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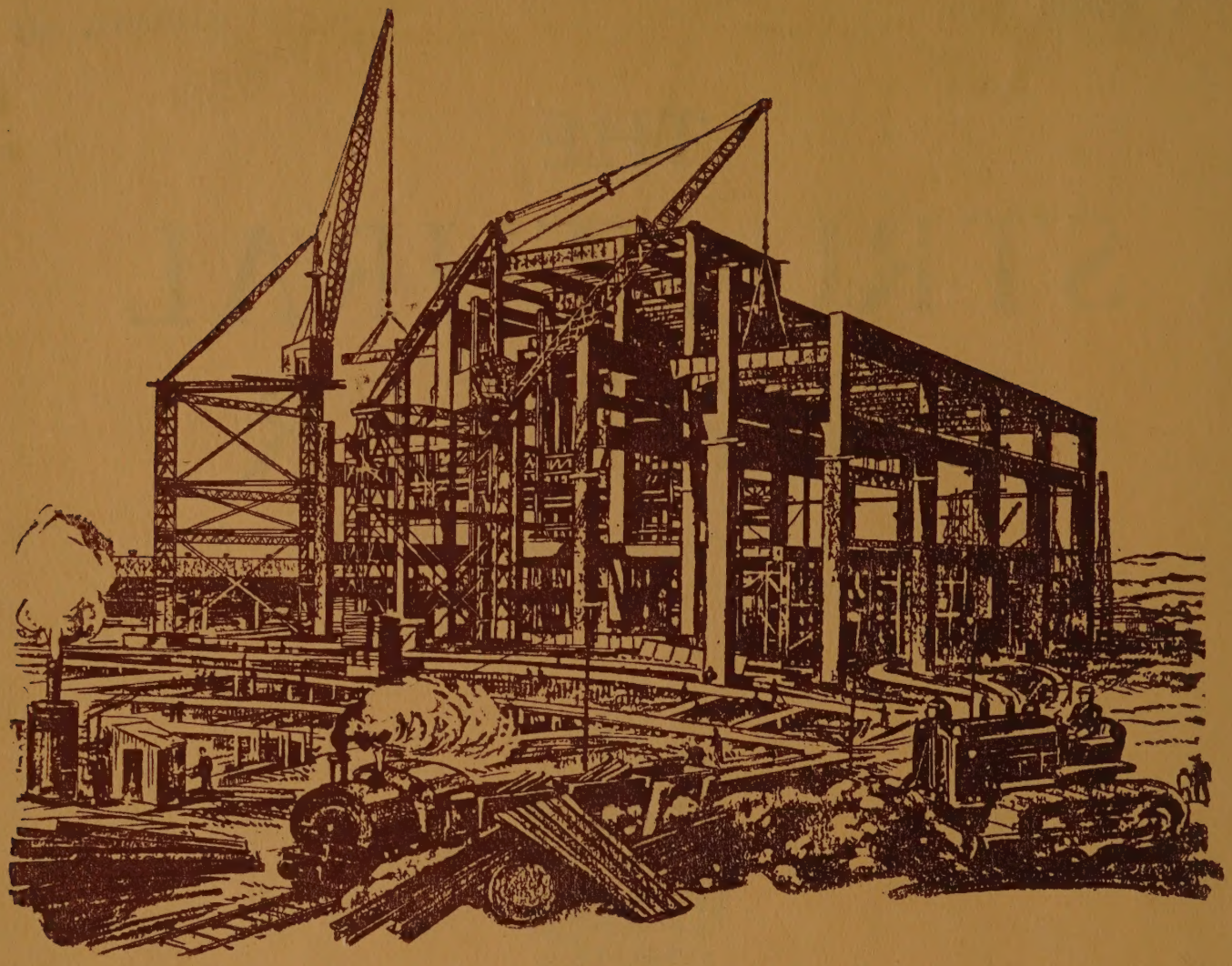

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Gasholder Development and Design*

By W. R. Garrett, A.M.I.C.E., A.M.I.Struct.E. (Associate Member of Council)

GASHOLDERS date back to the days of A. V. Lavoisier when in about 1781, he invented a laboratory instrument consisting of an inverted bell which was free to rise and fall in a tank of water. Gas was stored under the bell and over the water, and as the rise of the bell could be calibrated to indicate the volume of gas, we have the origin of the word " Gasometer."

The early gasholders were square and were invariably housed in buildings, but with the inevitable increase in size, combined with the problem of building the tank, the more practical circular shape was adopted. Wooden tanks as used by brewers were quickly superseded by tanks constructed in cast iron, brick or stone. The bell was constructed from wrought iron.

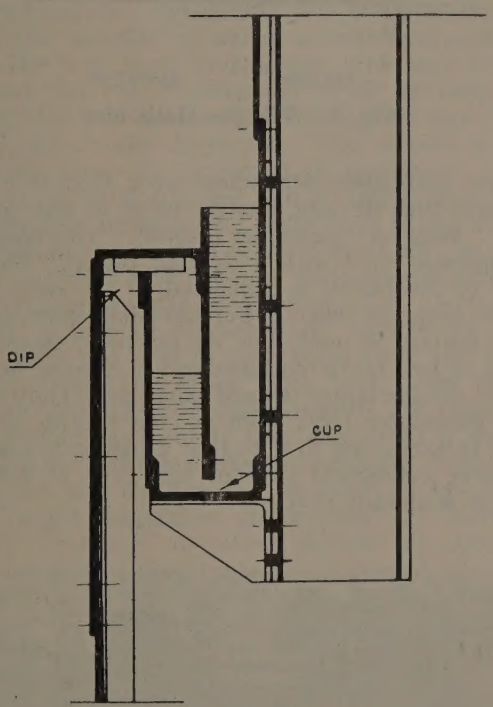


Fig. 1—Cup and Dip

The idea of constructing the bell in telescopic sections probably dates back to 1844, but it is fairly certain that the first having three sections or three lifts was built in 1852. To obtain a gastight joint between the sections, the inner section is provided with a cup at the bottom, and the next section is provided with an inverted cup or dip at the top, as shown in Fig. 1. The cup containing water from the tank engages with the dip of the next section which thus provides not only a mechanical link, but also the necessary water seal to prevent the escape of the contained gas.

All the early gasholders were constructed with external columns to guide and support the lifts, and this form of construction is known as a column guided gasholder, as shown in Fig. 2. Generally, the external guide framing extends to the full height of the bell, but in a few cases the framing is cut short, and we have what is known as a " flying " lift.

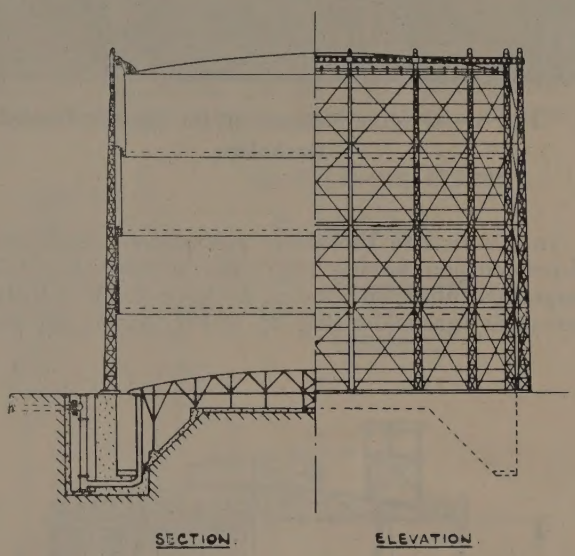


Fig. 2.—Column Guided Gasholder

In 1889 a wire rope guided gasholder was devised which dispensed with all external framing, but few working examples of this type are now available. The majority of gasholders built today are of the spirally guided variety which was invented in 1890. (Fig. 3).

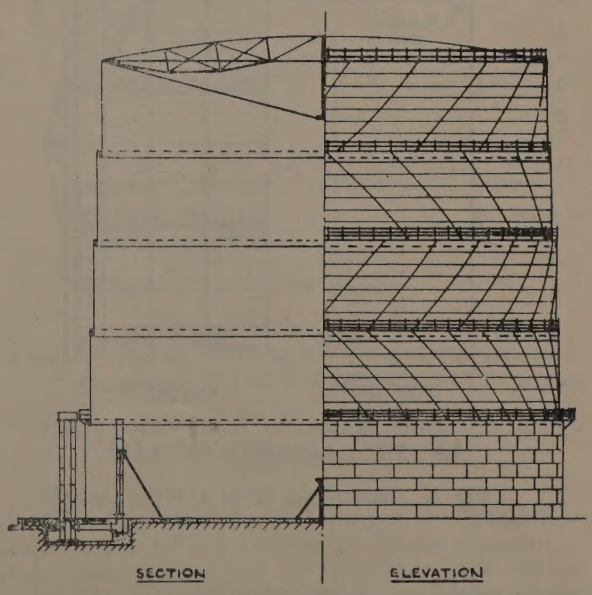


Fig. 3.—Spirally Guided Gasholder

This gasholder has special section flat bottom rails attached externally to the lifts at an angle of 45°, and carriages attached to the top of the next sections or lifts which guide and support each lift as it is inflated with gas. (Fig. 4.)

** Paper to be read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 13th January, 1955, at 6 p.m.*

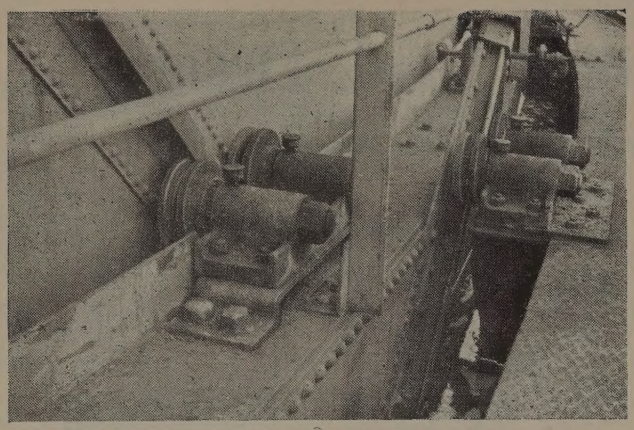


Fig. 4.—Guiding Mechanism for Spirally Guided Gasholder

In addition to the above gasholders, which are all dependent on having their lifts accommodated in a large tank filled with water, we have the M.A.N. tankless or dry gasholder (Fig. 5), which consists of a piston

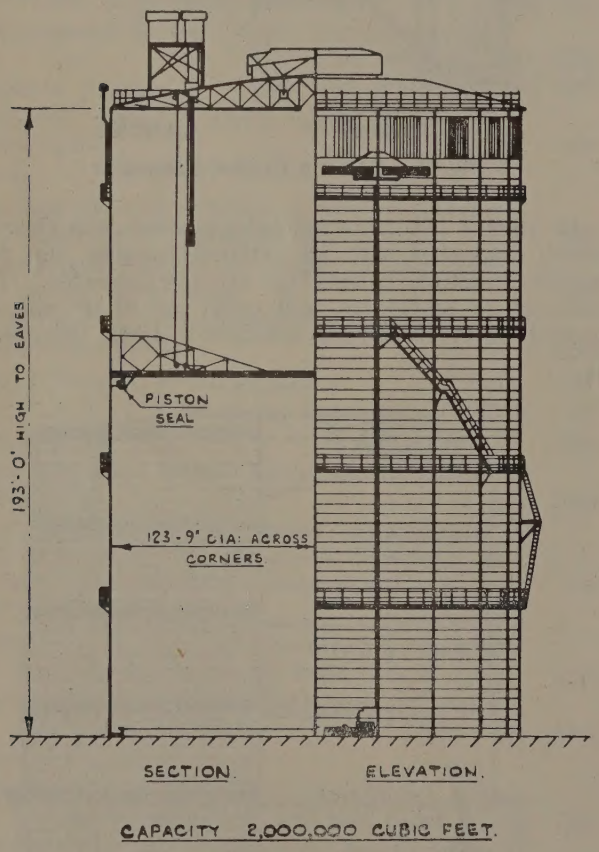


Fig. 5.—Waterless or M.A.N. Holder

working inside a polygonal shell. Gas for storage enters the cylinder and forces up the weighted piston. The piston is provided around its outer periphery with a flexible cup containing tar, and any leakage from the cup is collected at the base of the cylinder and returned by pumps to the cup. The first example of this type of holder was constructed in 1915, and today Chicago has one capable of storing 20 million cu. ft. of gas.

A similar gasholder to the above is the Klonnë holder which consists of a circular shell with a grease packed piston. This type dates back to 1927, and one constructed at Gelsenkirchen in 1938 with a diameter of 262′ 6″ and a height of 443′ 0″ had a capacity of 21 million cu. ft.

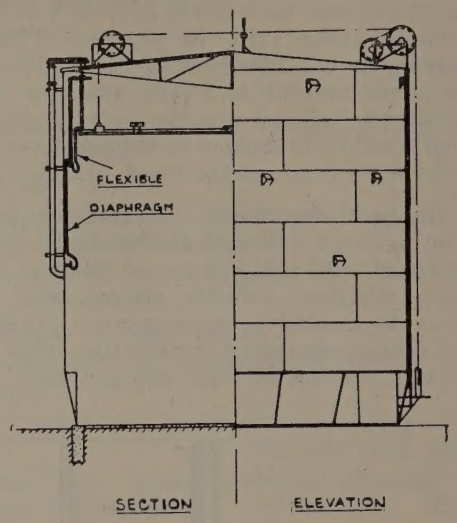


Fig. 6.—Wiggins Gasholder

Many gasholders depending on a flexible gastight seal have been devised, but the latest in this direction is the " Wiggins " gasholder (Fig. 6). This consists of a cylindrical shell with a floating piston. The periphery of the piston is connected to the shell with a flexible neoprene fabric. For the small sizes only one fabric curtain is used, but an ingenious device consisting of two fabric curtains and a circular fender is utilized for the larger storage capacities. Only nine of these gasholders have been erected in this country, and it is interesting to note that a patent to construct the outer cylindrical shell of this type of holder in concrete is already in hand.

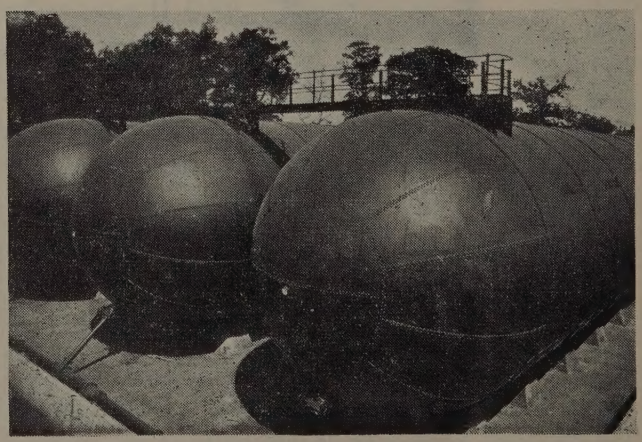


Fig. 7.—High Pressure Storage Holders

The main purpose of gasholders is for the storage of gas in order to overcome difficulties due to the varying rates of production and demand and the several types already described fulfill the requirements by storing the gas at low pressure, i.e. approximately 6″—16″ water gauge. Gas can also be stored in pressure vessels, but the high capital cost of the storage vessel plus high running cost for compressors can only be

justified in certain special cases. A sphere is the natural shape for a pressure vessel as it is only in this design that the stresses due to pressure are at a minimum. Despite the apparent advantages of a sphere over other shapes the horizontal cylinder with dished or semi-spherical ends, generally, provides the cheapest method of obtaining high pressure storage, as shown in Fig. 7. Underground storage is the logical step for the future, and investigations in this respect are being actively pursued by the Gas Industry.

Both column and spirally guided gasholders require a tank in which to work, and apart from total height of finished holder, it is immaterial whether the tank is above or below ground level. Cast iron, brick, stone and wrought iron tanks served a very useful purpose until more sutiable materials, concrete and steel, became available. Apart from the use of portland cement mortar, the first major use of concrete for constructing a gasholder tank was in 1872 for a tank 152′ 6″ diameter × 39′ 6″ deep. This was a "composite tank" consisting of a 9″ brick facing with a concrete backing. Header courses 1½ bricks thick were used every seventh course to bond the facing bricks into the concrete, and puddle clay backing was used behind the concrete to ensure a watertight construction. The first large all-concrete tank was built in 1875 and was 184′ 0″ diameter × 47′ 0″ deep. No backing clay was used and on filling with water some five to six cracks developed. The reason for the cracks was correctly diagnosed as insufficient consolidation of the earth backing, and in consequence nearly all subsequent underground tanks have incorporated a small percentage of steel reinforcement for the upper few feet. Reinforced concrete underground gasholder tanks became popular on the Continent about 1885. The first reinforced concrete tank constructed above ground level was built in America in 1905, and several partially buried examples have been built in this country. A small prestressed concrete gasholder tank above ground has recently been completed at Cromer. Underground concrete tanks are, generally, dearer than steel tanks with the necessary foundation, and, today, unless height restrictions are imposed by Town and Country Planning, or other factors are involved, economics dictate steel tanks above ground.

Whilst the design of steel tanks to withstand the fluid contents is an extremely easy matter, this is not the case with a buried tank constructed in brick or concrete. In brick and puddled clay tanks, the clay holds the water and the porous wall has only to be sufficient strength to support the clay and act as a retaining wall, i.e. the brickwork is in compression and is not subject to tensile stresses. On the other hand a concrete or brick tank internally rendered is subject to tension when filled with water and only subject to compression when empty. In the latter case, an accurate estimation of the earth backing is of prime importance for an economic design, and variation of the subterranean water pressure should not be overlooked. Where ground conditions are such that a partially buried tank is advisable, reinforced concrete with the working stress for steel limited to 12,000 lbs. per square inch and an allowable stress on the composite section limited to 175 lbs. per square inch, appears to produce a satisfactory design, but prestressed concrete is undoubtedly the present-day answer to this problem.

There are virtually no restrictions on the size of steel tank that can now be erected. The largest steel tank in this country is that built at Sheffield in 1938 for an 8 million cu. ft. capacity spiral gasholder in which the bottom tier of tank sides is 2″ thick and has a joint efficiency of 93.8 per cent. The tank is about 250′ 0″ diameter × 45′ 9″ deep. The minimum thickness of plate for a riveted or welded tank is usually restricted to ¼″ or 5/16″ for the upper tiers and bottom plates. The tendency to pare down the thickness of tank sides by taking the pressure on a tier of side plates some 6″ to one third up the depth of the tier is to be deprecated, especially if the working stress is also high. Fig. 8 indicates a graphical solution for determination of thickness of side plates for a tank 113′ 0″ diameter × 30′ 3″ deep, designed for a working stress of 7 tons per square inch. The stress at the tank bottom is calculated (in this particular case 3.95 tons per square inch) and extended proportionately up to 7 tons per square inch. A second scale can now be constructed so that the 7 tons per square inch coincides with a 1″ thick plate, i.e. representing 100 per cent. joint efficiency. New scales can be formed to give any desired efficiency,

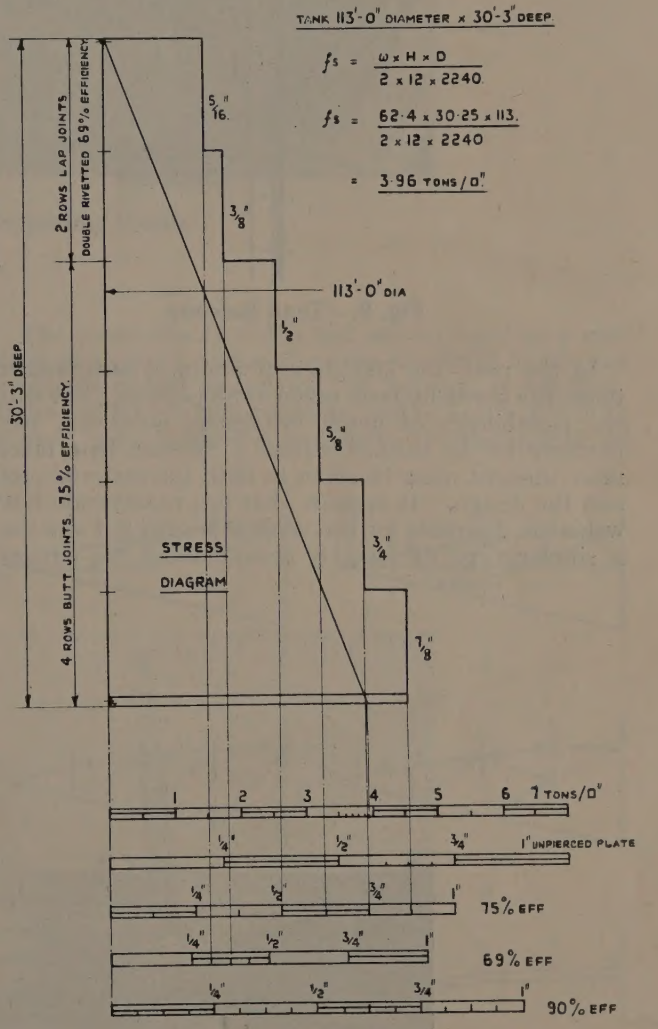

TANK 113'-0" DIAMETER x 30'-3" DEEP
$f_s = \frac{\omega \times H \times D}{2 \times 12 \times 2240}$
$f_s = \frac{62{\cdot}4 \times 30{\cdot}25 \times 113}{2 \times 12 \times 2240}$
= 3·96 TONS/□"
2 ROWS LAP JOINTS DOUBLE RIVETTED 69% EFFICIENCY
4 ROWS BUTT JOINTS 75% EFFICIENCY
30'-3" DEEP
113'-0" DIA
5/16"
3/8"
1/2"
5/8"
3/4"
7/8"
STRESS DIAGRAM
1 2 3 4 5 6 7 TONS/□"
1/4" 1/2" 3/4" 1" UNPIERCED PLATE
1/4" 1/2" 3/4" 1" 75% EFF
1/4" 1/2" 3/4" 1" 69% EFF
1/4" 1/2" 3/4" 1" 90% EFF

Fig. 8.—Graphical Solution of Thickness for Tank Plates

and these sub-divided will indicate the thickness of plate required to fall outside the stress diagram. It will be observed that had a 90 per cent. joint efficiency been adopted for the lower three tiers of plates these could have been decreased in thickness from ⅞″, ¾″ and ⅝″ to 11/16″, 19/32″ and ½″ respectively. Joint efficiencies for double riveted lap joints approaching 75 per cent. are used on the thinner plates, but for anything in excess, inside and outside butt covers are required, and joint efficiencies in excess of 90 per cent. are obtained.

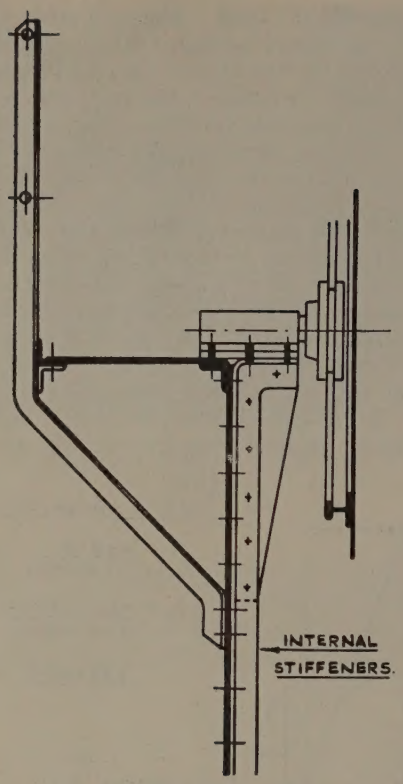


Fig. 9.—Tank Balcony

In the past, the efficiency of joints in butt welded tanks has normally been taken as 85 per cent., but with the possibilities of notch brittleness occurring, particularly in the thicker plates, it appears that much more thought must be given to both the material used and the design. It appears that full penetration butt welds are desirable for the vertical seams, but whether a similar type of joint is desirable for the circumferential seams is doubtful. Lapped circumferential seams with comparatively light fillet welds may well prove the best form of construction. Present prices indicate little difference between riveted and welded tanks, but if for the latter special materials and X-ray examination are essential to avoid a notch brittle structure, then riveted tanks with the attendant difficulties of obtaining good riveters may well be the ultimate answer. The only other alternative is to reconsider the numerous designs which have been adopted in the past. One such scheme, patented in 1897, avoids the rapid increase of thickness required for the lower tier of plates by encompassing a tank of

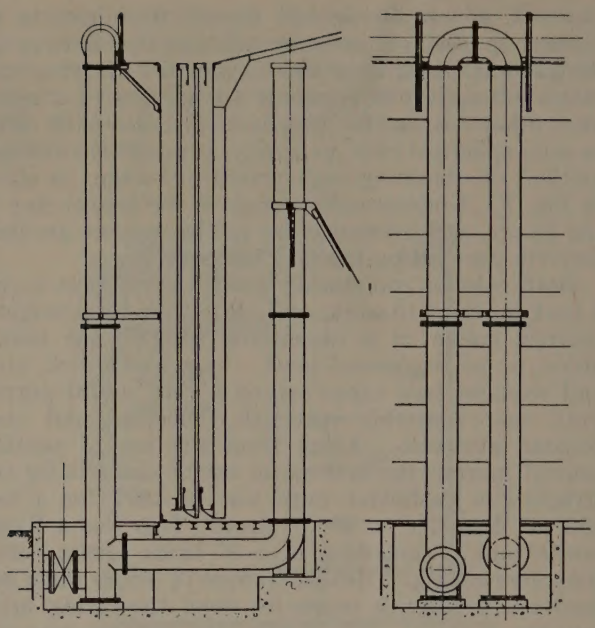


Fig. 10.—Inlet and Outlet Pipes

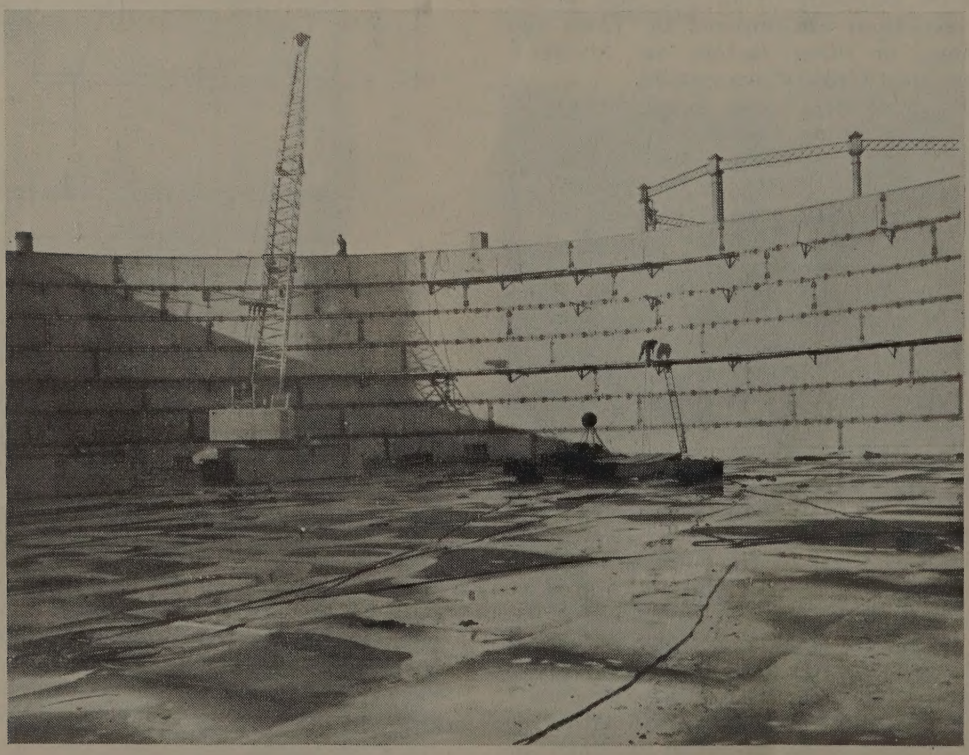


Fig. 11.—Photograph of Portable Erection Crane

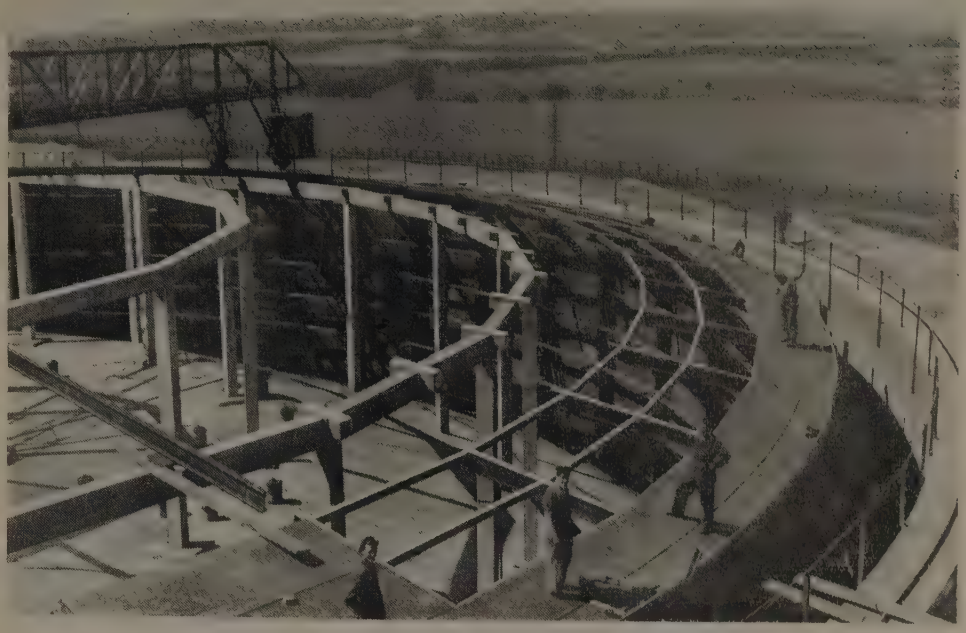


Fig. 12.—Tank Crown Supporting Frame

uniform plate thickness with a slightly larger tank of uniform thickness, but of half the height, the annular space so formed being filled with water. Tanks using buckled, bulged or radius walls all avoid the use of thick bottom tier plates.

Gasholder tanks are similar in all respects to tanks required for fluid storage requirements, except that the former have rest blocks for supporting the lifts when at rest, and a chequer plate balcony which provides access and gives rigidity to the upper edge of the tank, as shown in Fig. 9. In column guided gasholders, the tanks are provided with vertical channel guide sections, and, in spiral holders, supports are arranged to transfer the load from the spiral carriages to ground level. Gas inlet and outlet pipes are usually arranged some distance apart to ensure a certain degree of mixing. With tanks below ground there is little chance of flooding the district mains with water, but, in the case of steel tanks, this possibility is removed by looping the gas mains above tank water level prior to their entry into the inlet and outlet pipes, as in Fig. 10.

The load imposed on the foundations of a holder mainly consists of the weight of water contained in the tank, and a reinforced concrete slab to bridge any local weakness is often all that is required. Should the subsoil be incapable of carrying the superimposed load, piles at about 6′ 0″ — 7′ 0″ centres with beams and slabbing are provided.

Steel tanks of both welded and riveted design are normally erected and tested with the first tier of side plates clear of the ground, and subsequently lowered after testing. To erect the tank on its foundation and thus omit the underside fillet joint on the lapped bottom plates and finally test by a vacuum box has its advantages, but it is doubtful whether this is an advisable form of construction for gasholders. Erection of tank side plates is usually accomplished by a crane, or travelling derrick, working on a rail track fixed to the bottom plates, although the pneumatic tyred portable crane appears to do this job just as efficiently. (Fig. 11.)

The crown sheets of the bell are arranged as a segment of a sphere, and as these are supported by the gas when the holder is inflated, no trussing is required. However, for erection purposes, and when the holder is at rest, some support is necessary. In the case of an

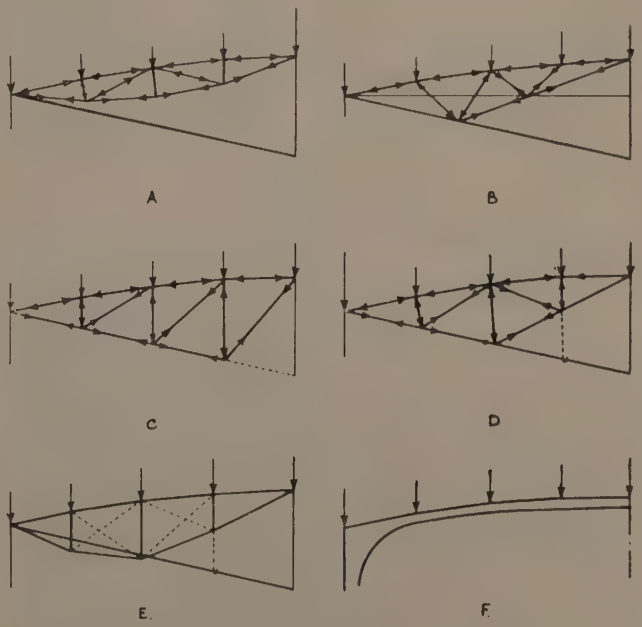


Fig. 13.—Crown Trusses

underground tank, which is almost invariably built with a dumpling, this can be most economically arranged as a fixed timber, steel or reinforced concrete structure (Fig. 12), but for a steel tank, where the supporting columns are long, a truss framing with a centre support is the usual solution. (See Fig. 3.) If the holder is required to throw a heavy pressure, a trussed crown framing adds weight to the bell and thus serves two functions. Crown framing is designed to

carry the dead weight of top sheets, snow and erection loads and Fig. 13 indicates a few of the many designs which can be adopted. The bottom chord in Fig. 13E follows the bending moment stress diagram for the loads

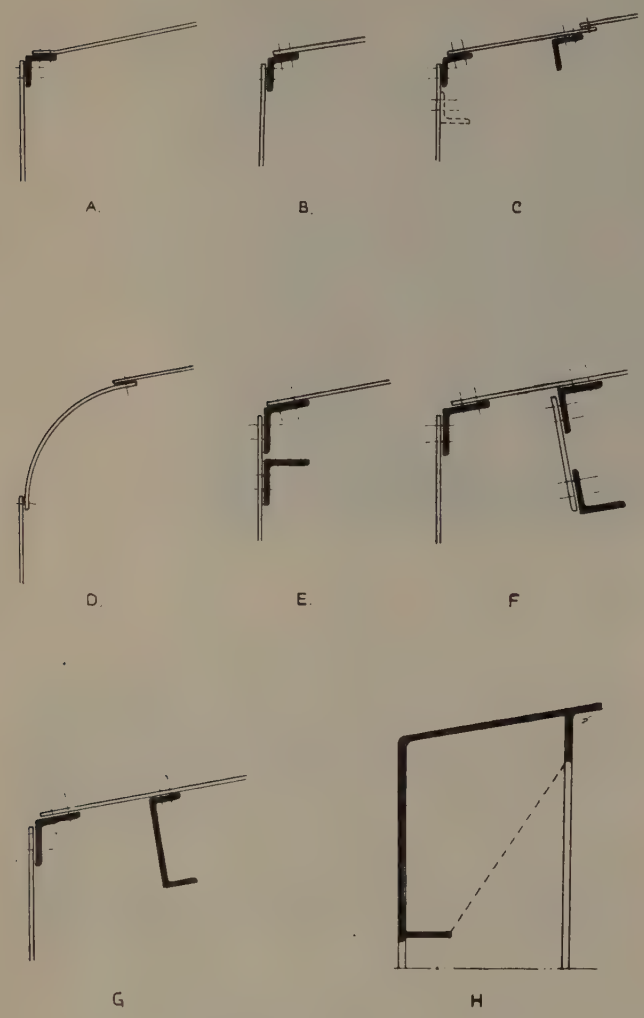


Fig. 14.—Curb Designs

to be carried and the main tie is arranged as two flats which embrace and provide lateral stiffness for the truss. Fig. No. 13F is a portal design. The crown sheets are attached to the vertical side sheeting of the lift by a top curb which carries a heavy compressive force. For a small holder, an angle swept to radius and opened to suit the rise of crown is suitable, but for the larger holders a compressive load of several hundred tons may be involved, and this necessitates a fairly heavy construction. A few of the many designs of top curbs adopted for gasholders are shown in Fig. 14. The trussed crown materially assists in the design of the top curb. Design requirements necessitate crown sheets of some 10 G. or 3/16″ thick, whereas the side sheets of the lifts will be about 10 G. Hence, it is necessary to thicken up the top and side sheets gradually as they approach the heavy sections comprising the top curb. Vertical stays are provided for all lifts, and these, connected to the cup and dip sections, form a circular framework on which to support the side sheets.

In the case of guide framed gasholders, the wind load on the side of the bell is transmitted to the framework by the guide rollers. Various arrangements of guide rollers, i.e. tangential, skew etc., in theory, give a better stress distribution to the framework (Fig. 15), but in actual practice it is wise to rely only on the radial rollers. The photograph (Fig. 16) shows a guide framed gasholder in course of erection and three of the set of four roller carriages erected. Diagonal bracing is usually provided for the vertical standards with horizontal girders to give lateral stiffness. An important feature, for this type of holder, is to supply a deep

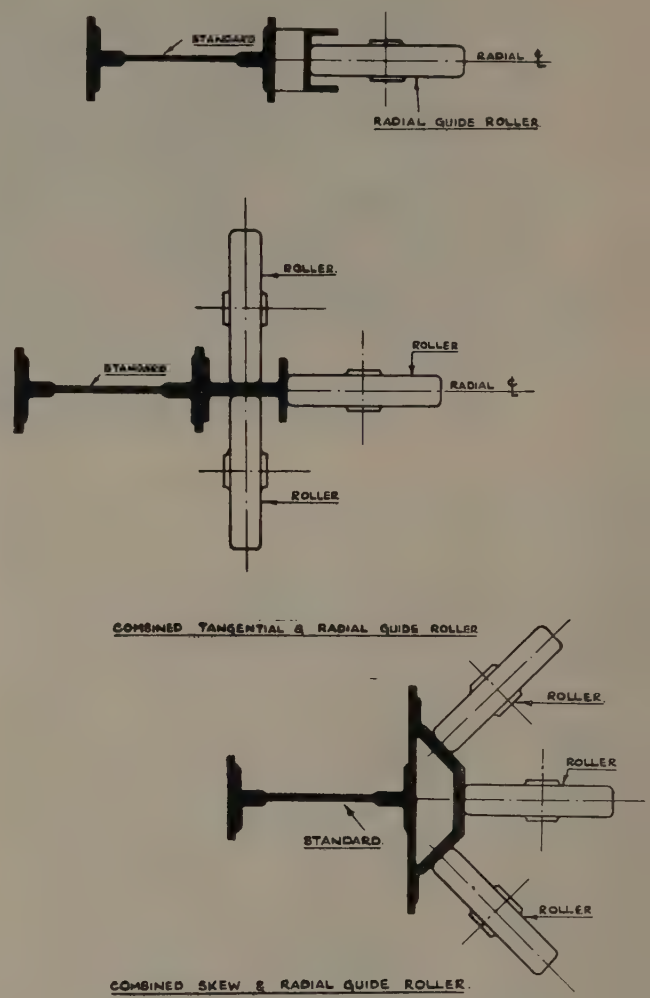


Fig. 15.—Design of Column Guide Rollers

wind girder at the top of the framing to ensure circularity of the upper ends of the cantilever standards.

The bell of the spirally guided holder is similar, in many respects, to that of its column guided counterpart. The main difference is the application of the external guide rails which brace the lift in a diagonal direction. These guide rails vary for the large holders from a 70 lb./yard B.S. flat bottom rail section to special sections of 55, 41 and 40 lbs./yard for the smaller holders. Generally, the maximum pitch of the rails is about 30′ 0″, and rarely exceeds the depth of the lift. The pitch is determined by the working load on the carriages, and, wherever possible, it is usual to arrange the number to be a multiple of four. Thus, for a medium size four lift holder of, say, 2 million cu. ft. capacity, the number of spiral rails might be 16, 20, 24 and 32 for the inner, second, third and outer lifts respectively. The inner lift is usually provided with two vertical stays to each spiral rail, but the number of stays to the remaining lifts is dependent on design requirements, and is usually just under two per rail. There is a wide variation in the design of guide

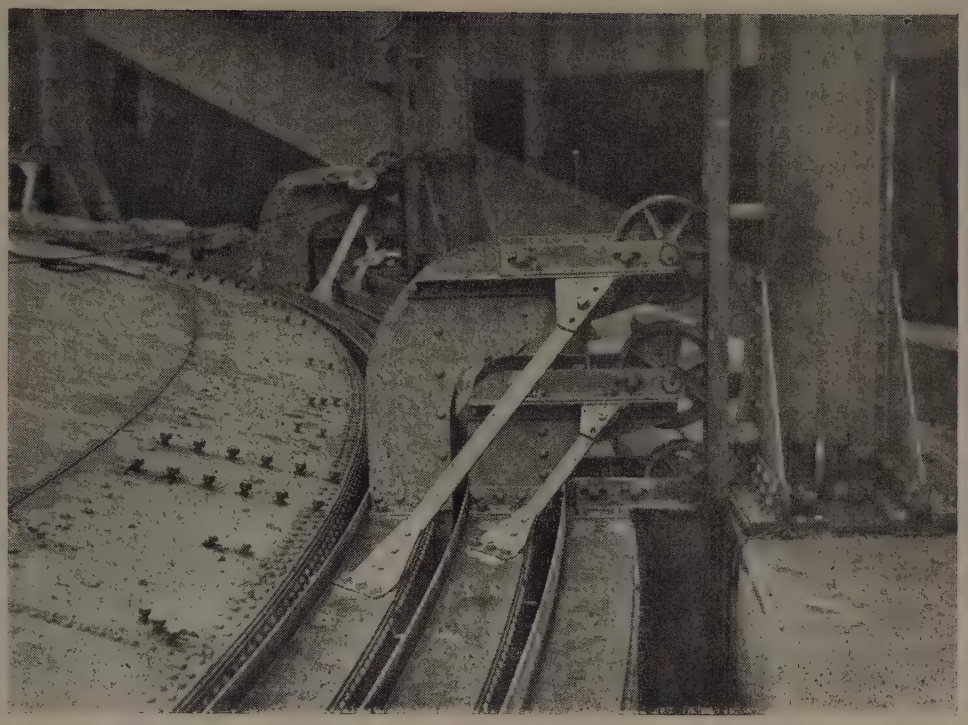


Fig. 16.—Radial Rollers for Column Guided Gasholders

carriages, but, generally, the two wheel variety with a tread diameter of 10″ is fairly common. The wheels are usually cast steel or mild steel. The axle diameters vary from 2″ up to 4″ diameter, and these may be ordinary mild steel or carbon steel.

Apart from the self weight or dead load the principal loading for a gasholder is that produced by wind and snow. The latter, when evenly distributed over the crown sheets, merely increases the gas pressure and the stress in the top curb, and does not affect the framing. However, to safeguard against the distinct possibility of uneven snow distribution over the crown it is wise to allow for such a contingency. A reasonable snow allowance is 5 lbs. per square foot, acting over one quarter of the crown area on the leeward side. The wind pressure increases with height above ground level, but the most practicable course is to use some uniform figure. Most authorities assume wind values of 40 lbs. per sq. ft., but this, for such a large structure, appears excessive, and 32 lbs. per sq. ft. would seem to be more reasonable. Reduction in wind pressure due to shape has been variously estimated from 41 per cent. to 57 per cent., but again 50 per cent. is reasonable. Maximum working stresses are those usually adopted for steel structures.

In column guided gasholders, it is assumed that only the radial carriages on the leeward side that are less than 60° with the direction of the wind are effective. (Fig. 17A.) The load carried on each carriage is proportional to the distance from the neutral axis, i.e. cosine of respective angle with the direction of wind, and the resistance is equal to the distance from the neutral axis. Thus, the relative effective resistance of the carriages will be the distance from the neutral axis squared, or cosine squared. In addition, the top carriages of the inner lift are assumed to take the whole eccentric load caused by snow (Fig. 17B). Thus, the total wind load on each lift is proportioned to the effectiveness of its carriages and a series of shear loads on the framing is obtained. A small additional shear load is added to cover the wind on the framing. The framing acts as a hollow cylindrical cantilever. The horizontal component of the wind which each bay receives is proportioned to the cosine of the angle that the bay makes with the direction of wind (see windward side of diagram Fig. 17A) and, in addition, its resistance is proportioned to the cosine of the angle of the bay in relation to the wind. With the maximum loads on any bay of framing the members can be determined (Fig. 17C). To avoid excessive deflection of the whole framework large gusset plates should be used, and it is wise to limit the stress in diagonal members to about 5 tons per sq. inch.

Column guided gasholders are rarely built today, but by adopting welding, and a rigid frame design, it is possible to save some 30 per cent. of the weight of the framing of a normal commercial design.

The cost of a spirally guided gasholder is generally less than that of the column guided type, and this, combined with its proved reliability, has resulted in the comparatively rapid increase in the number of spiral holders now in service. The design principles are more readily appreciated if actual figures are taken and in the following example the design of a one million cubic feet capacity holder will be briefly investigated. The lift sizes for this capacity will be :

	Diameter		*Depth*	*Number of Spirals*
Inner lift	117′ 9″	×	30′ 0″	12
Middle lift	120′ 4″	×	30′ 0″	16
Outer lift	122′ 11″	×	30′ 0″	24

and these will be accommodated in a steel tank 125′ 6″ diameter × 31′ 0″ deep. (Fig. 18.)

With the exception of the two carriages which occur on the neutral axis it is assumed that all carriages, by taking either upward or downward loads, contribute to the stability of the holder. Their effectiveness is proportional to the distance from the neutral axis, i.e.

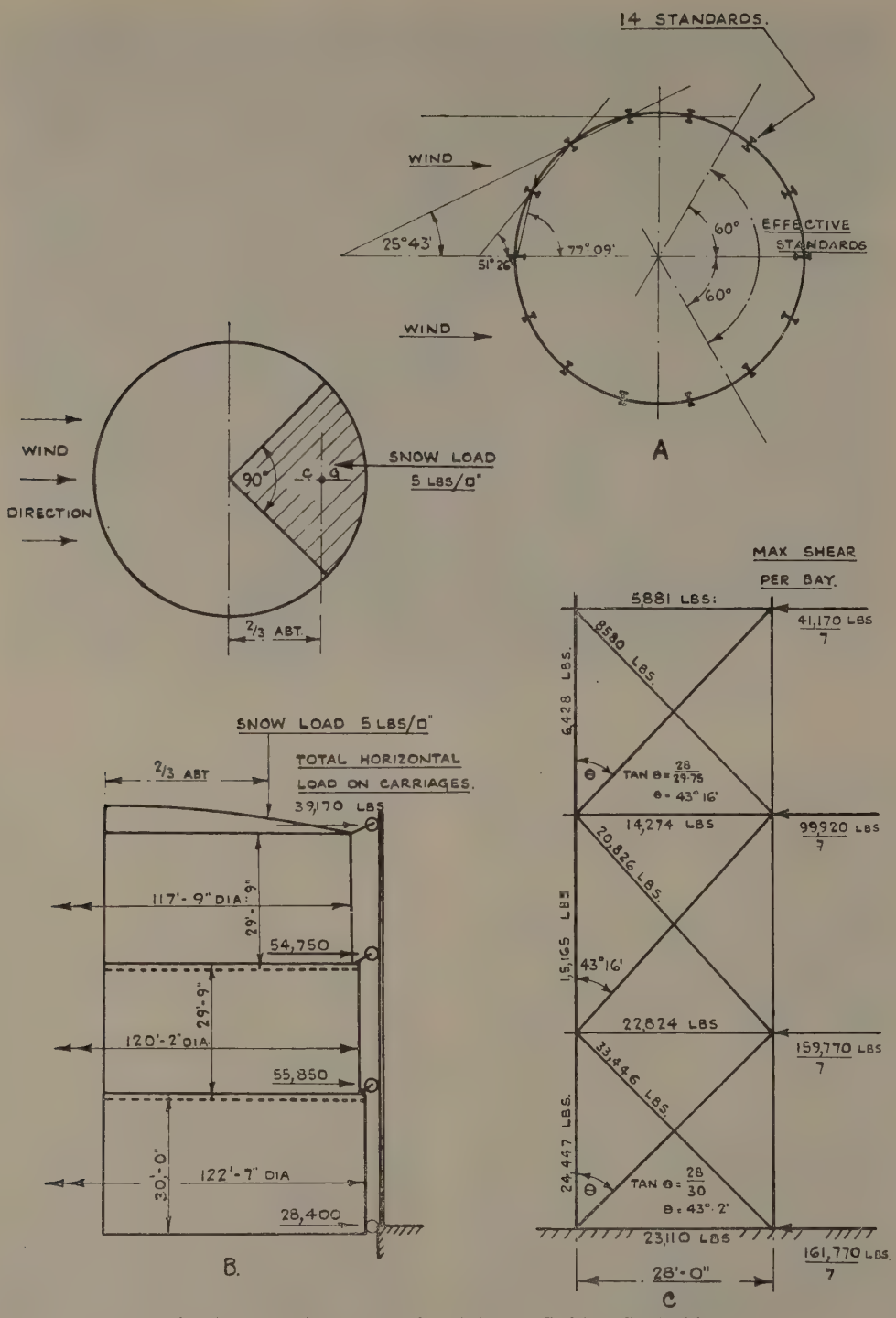


Fig. 17.—Design Loads for Column Guided Gasholders

cosines of respective angle to the wind direction and their resistance will be equal to their distance from the neutral axis. Thus, the relative effective resistance of the carriages will be the distance from the neutral axis squared i.e. cosine squared. Assuming the holder is fully inflated to give maximum load conditions on the 24 carriages situated on the tank balcony the relative effective resistance of carriages will be :

Angle	*Number of Rollers*	*Cosine of Angle*	*Number of Rollers × Cosine² of Angle*
0°00′	2	1.00	2.000
15°00′	4	0.9659	3.731
30°00′	4	0.8660	3.000
45°00′	4	0.7071	2.000
60°00′	4	0.5000	1.000
75°00′	4	0.2588	0.269
90°00′	2	0.0000	0.000
			12.000

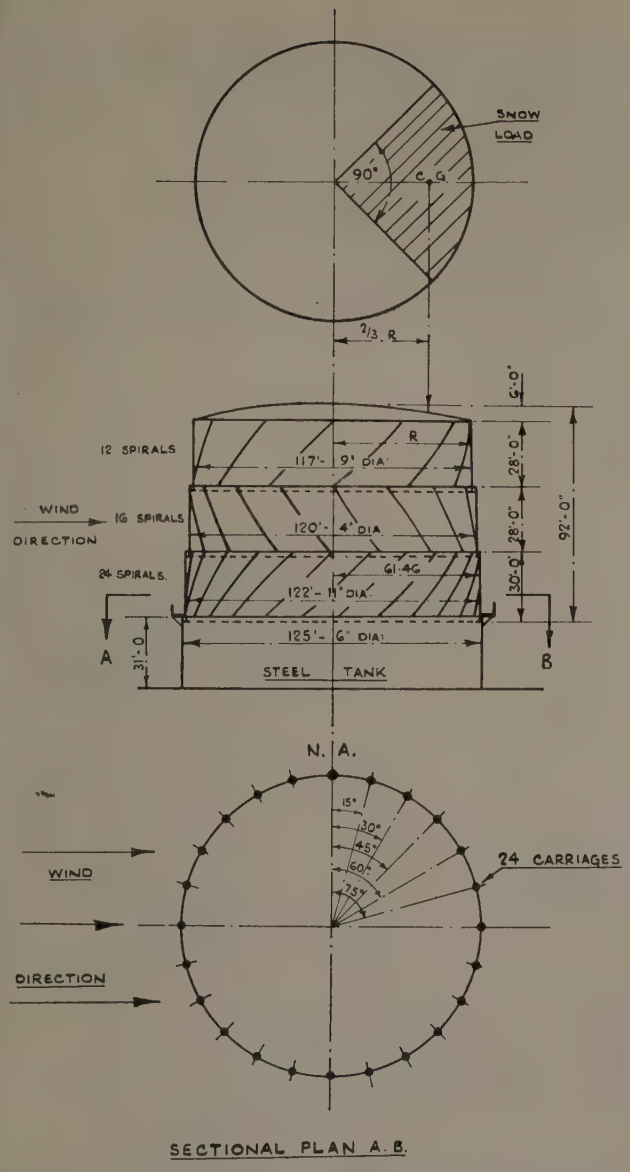


Fig. 18.—Design Loads for Spirally Guided Gasholders

This means that although there are 24 carriages the maximum vertical load carried by any one is equal to one twelfth of the total load or if there are x carriages the maximum load on any one is the total load divided by $0.5x$.

The total vertical load on the carriages will be derived from (*a*) wind on side, (*b*) snow on crown.

(*a*) Taking the horizontal wind load of 32 lbs. per square foot on a normal surface or 16 lbs. on the diametrical projection of a cylindrical surface, the equivalent total vertical load in tons on the tank carriages will be :

$$\frac{\text{Mean diam.} \times \text{Total height} \times \text{wind pressure} \times \text{lever arm}}{2240 \times \text{radius of outer lift}}$$

$$= \frac{120.33 \times 92 \times 16 \times 46}{2240 \times 61.46}$$

$$= 59.2 \text{ Tons.}$$

(*b*) It will be assumed that a snow load of 5 lbs. per square foot will act on one fourth of the area of the crown and this, to give maximum bending moment, will be positioned on the leeward side of the holder. The centre of gravity of the snow load acts at a point approximately two-thirds of the distance from the centre of the crown to its outer edge.

$$\text{Snow load} = \frac{\text{Area of Crown} \times 5 \text{ lbs.}}{4}$$

Vertical load on carriages (from snow)

$$= \frac{\text{Area of Crown} \times 5}{4 \times 2240} \times \frac{\text{Two-thirds radius of inner lift}}{\text{radius of outer lift}}$$

$$= \frac{10890 \times 5}{4 \times 2240} \times \frac{2 \times 58.87}{3 \times 61.46}$$

$$= 3.9 \text{ Tons.}$$

Total maximum vertical load on tank carriages

$$= 59.2 + 3.9 = 63.1 \text{ Tons.}$$

$$\text{Maximum carriage load} = \frac{63.1}{12} = 5.26 \text{ Tons.}$$

The vertical load of 5.26 tons is resisted by one of the two carriage rollers (Fig. 19), and owing to the 45° inclination of the spiral rails the resultant roller load is $5.26 \times 1.414 = 7.44$ Tons. The bending moment on the axle is divided by the section modulus in order to determine the maximum stress on the axle.

The carriage loads on the top of the outer lift and top of the middle lift are obtained in a similar manner

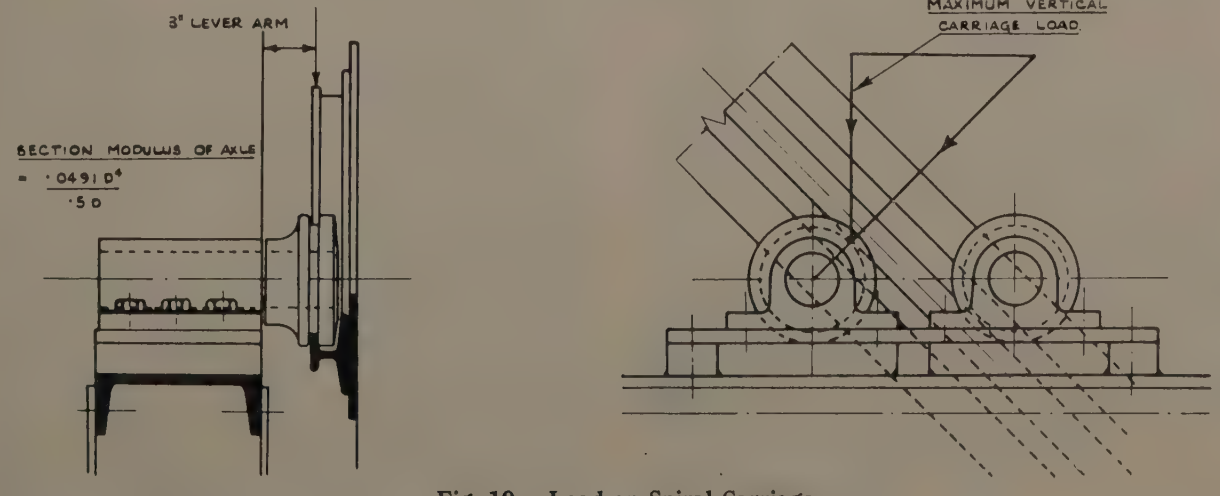


Fig. 19.—Load on Spiral Carriage

to the above, the overturning moment due to wind being calculated when two lifts or one lift respectively are inflated.

Maximum stress occurs in the crown and side sheeting when the holder is fully inflated ; the former is designed as a portion of a sphere subject to internal gas pressure and the latter as a cylinder subject to a similar force. The theoretical thickness of crown sheets is approximately double that of the sides. The top curb is designed to resist the pull in the top sheets, the weight of the side sheets, the internal gas pressure and the external wind pressure. The inner lift stays are designed to take the weight of the crown framing and side sheets, plus snow when the holder is at rest. The stays for the remaining lifts take a proportion of the maximum carriage loads depending on number in relation to carriages and also have to be capable of carrying the wind load as a beam. The cup and dip rings have to take the compressive wind load but practical requirements are such that the working stress in this particular portion of a holder is usually fairly low.

Many interesting details of design occur in this type of structure, but it is felt that sufficient information has been given to appreciate the general principles and problems which are met in gasholder construction.

Correspondence

The Institution, whilst being at all times pleased to open its columns to correspondence, cannot accept responsibility for the opinions expressed.

To the Editor of THE STRUCTURAL ENGINEER

Autogenous Healing of Concrete in Compression*

E. F. WHITLAM, M.Sc.(Eng.), A.M.I.C.E., A.M.I.Struct.E.

Sir,—I have read, with great interest, the Paper on Autogenous Healing of Concrete by Mr. E. F. Whitlam, but think his conclusions are open to question.

Having obtained a thoroughly satisfactory face on the old concrete why does he wish to plaster it with grout and let this dry out before the new concrete is placed? However thin the coating of grout there are now two joints liable to crack instead of one. The old concrete is drying out at one rate, the grout at another and the fresh concrete at yet another.

Most Specifications go into great detail regarding the materials and mixes to be used for the concrete required but only in rare cases is the grout given the same consideration. In most cases some purely arbitrary figure—1 : 1, 1 : 2 or 1 : 3—is selected irrespective of the concrete mix or even if the joint is between pre-cast and in situ units.

Practical experience seems to show that unless the mix of the grout is varied to suit the various concretes to be jointed the result will be unsatisfactory and that rich grouts are very much more liable to give cracked joints than are lean ones.

Many Specifications call for joints to be grouted immediately preceding the pouring of the new concrete. With horizontal joints in concrete this means that the grout is immediately absorbed by the new concrete and appears to be a waste of time and material because the same result is obtainable, and much more easily, by slightly increasing the cement in the first few pours. With vertical joints the conditions are somewhat different because it is difficult to keep the joints damp until the new concrete is consolidated without getting excessive water in the new concrete. In this case the grout is advantageous because it acts as a lubricant during consolidation.

Whilst Mr. Whitlam's experiments confirm that cracks can be closed by water I would suggest that if they had been carried further by using grout it might be possible to get a line on grout mixes which would reduce the present site difficulties of obtaining good joints free of cracks.

Yours, etc.,

H. WILMOT BARRAS (Member)

Selsey,
Sussex.

15th September, 1954.

The Author writes :—

I should like to thank Mr. Barras for his comments. Perhaps the point was not made quite clear enough regarding treatment of the concrete face. I stated that the grout should be allowed to 'go off.' The main purpose of this was to ensure that the old concrete was not running with water and I would think 20—30 minutes a suitable time. It was never intended that it should be allowed to dry out. I entirely agree that under such circumstances it would be liable to crack. Many specifications do not give mixes for grout although the present tendency is to do so, particularly in the case of stressed concrete work. It is agreed that the mix should bear a relation to the concrete. Normally a 1 : 3 or 1 : 3½ corresponds fairly closely to 1 : 2 : 4 concrete. Is it a waste if the grout is absorbed by the new concrete? Surely one of the functions is that it shall provide an additional amount of cement at the raw concrete face and it is doubtful if such a thin layer is removed by the new wet concrete, even though it absorbs the majority of the grout.

To increase the cement content gives rise to various difficulties, apart from the fact that it is not usually desirable to involve repeated changes of mix on site.

I am of the opinion that with correct supervision and suitably trained men, new concrete can be placed direct against the old concrete, providing the surface of the old is just moist but not wet. The resulting joints need scarcely be visible, and such results have been obtained when I have carried out the mixing and placing, admittedly under reasonable conditions, on my own account. Until the time when all concreting gangs undergo full training it is doubtful if they can be relied upon to do this and presumably for the present the use of a grout will stay.

It has been pointed out to me that one important reference has been omitted.

I think that attention should be drawn to an article in the February 1937 issue 'Concrete and Constructional Engineering' by Mr. Leslie Turner (Past President). This gave details of tests carried out on Tensile Healing of Concrete and showed the recovery of briquettes tested at 1, 3, 7 and 28 days and healed for periods of up to six months.

**The Structural Engineer*, Vol. XXXII, No. 9, pages 235-243, Sept., 1954.

Brittle Fracture Problems in Steel Construction*

By G. M. Boyd, M.I.N.A., A.M.I.Struct.E.

Introduction

THE problem of brittle fracture in mild steel, which is normally regarded as a ductile material, has been with engineers for many years, whether they have realised it or not. A recent review by M. E. Shank[1] cited cases as far back as 1886, and many other cases are reported in the engineering literature. A still greater number have probably not been reported publicly.

The existence of the problem has not, however, been generally recognised among practising engineers, although it has been well known to some metallurgists and physicists. The reasons for this are probably manifold, but it seems likely that the most potent has been the engineer's natural reluctance to believe that a material which shows something like 20 per cent. elongation and 50 per cent. reduction of area in a tensile test, even at low temperatures, could possibly be regarded as brittle. When brittle fractures occurred in service, they were usually ascribed to over-stressing, shock, or defects in material, design or workmanship. It was only vaguely realised that low temperatures tended to embrittle steel, and this was reflected in some empirical dodges used by fabricators, such as the practice of " taking the chill off " steel plates before punching or shearing them in frosty weather.

Of recent years, however, the problem has been brought very forcibly to the attention of engineers by some highly spectacular and catastrophic fractures in steel structures, particularly ships,[2] bridges,[3,4] oil storage tanks,[1,5] and other containers.[6] Some examples of such fractures are shown in Figs. 1, 2, 3, 4 and 5. Many of these have been in welded structures, but it is important to observe that brittle fractures have not been confined to welded structures, and indeed many of the failures which have afflicted riveted structures in the past are now recognised as being due to the same phenomenon. One might even go so far as to say that the majority of serious fractures which have occurred in steel structures generally, have been of the brittle variety. It is probably true, however, that in welded structures the fractures have been relatively more frequent and more extensive. More frequent, probably because welding tends to produce conditions which favour the initiation of such fractures, as will be seen later. More extensive, probably because the inherent continuity of welded structures provides fewer barriers to the progress of fracture.

Neither have these fractures been confined to any particular kind of steel structure. If sufficient search

Fig. 1.—View of a T-2 tanker after splitting in two at her outfitting dock[2]

Reproduced by courtesy of the Ship Structure Committee (U.S.A.)

* *Paper to be read before the Institution of Structural Engineers at* 11, *Upper Belgrave Street, London, S.W.*1 *on Thursday,* 27*th January,* 1955, *at* 6 *p.m.*

Fig. 2.—View inside crack showing complete fracture of longitudinal bulkheads[2]
Reproduced by courtesy of the Ship Structure Committee (U.S.A.)

were made, it would probably be found that no kind of steel construction was immune, but it is now known that the liability increases with the thickness of the material, so that the heavier structures, which are increasingly called for by modern requirements, are likely to be more susceptible.

KINDS OF FRACTURE

To most engineers, a fracture is just a fracture, and little attempt is generally made to distinguish between the many kinds. If this were done, and the ability to distinguish were more widespread, the true causes of many failures might be more accurately assessed, and their repetition avoided.

Before describing the kind of fractures with which we are here concerned, it is desirable to describe a few other types, in order to distinguish them from those which are the subject of this paper.

FATIGUE CRACKING : This type of cracking, which has been dealt with rather fully by Dr. R. Weck in a recent paper before this Institution,[7] occurs in structures subjected to fluctuating loads. The cracks appear without any measurable plastic deformation, and spread slowly until the member is so weakened that it ruptures by some other mode, such as those which occur under static overloading. The faces of true fatigue cracks, when new and clean, usually appear smooth and polished, but show faint markings indicating successive positions of the progressing front. Quite often the final rupture occurs suddenly, and in a brittle manner, the surfaces showing a sparkling "crystalline" texture. Such a crystalline break, even when forming the final stage of fatigue, is not strictly speaking a fatigue fracture, but is a brittle fracture of the kind which will be

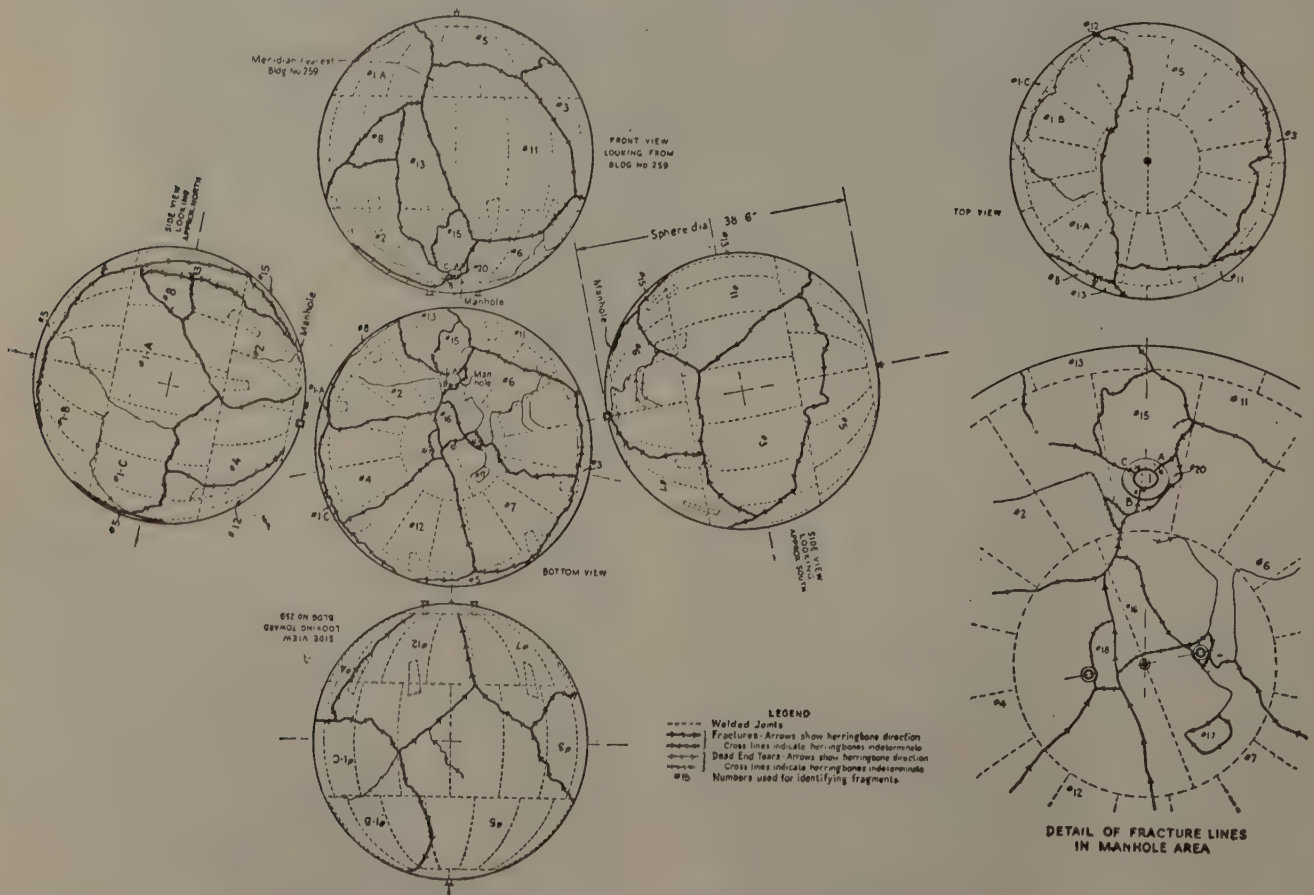
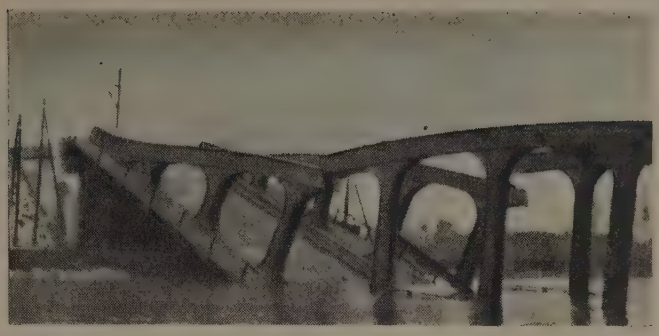

FRONT VIEW LOOKING FROM BLDG No 259
Sphere dia 38 6"
Manhole
BOTTOM VIEW
TOP VIEW
LEGEND
Welded Joints
Fractures - Arrows show herringbone direction
Cross lines indicate herringbones indeterminate
Dead End Tears - Arrows show herringbone direction
Cross lines indicate herringbones indeterminate
Numbers used for identifying fragments
DETAIL OF FRACTURE LINES IN MANHOLE AREA

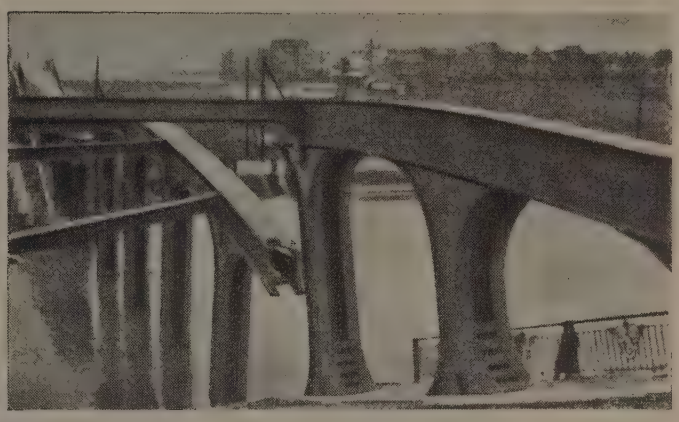
Fig. 3.—Failure of a hydrogen sphere at Schenectady, U.S.A.[6]
Reproduced by courtesy of *The Welding Journal* (U.S.A.)

Bridge Collapsed in Three Parts



View from Hasselt Towpath

View looking towards Hasselt



View from top of Hasselt Abutment

Fig. 4.—Failure of Hasselt Bridge[3]

Reproduced by courtesy of *Engineering*

dealt with later. " Fatigue Cracking " is a term which, strictly, should be applied only to the first stage, before the final rupture.

STRESS-CORROSION CRACKING : This occurs under static or fluctuating stress in the presence of a corrosive agent. The chemical action weakens the boundaries between the crystals of the metal, and cracking progresses slowly under the action of the stress until the member is sufficiently weakened to fracture by some other mode. Such stress-corrosion cracks are usually " intercrystalline," which means to say that the breakdown occurs on the boundaries between the crystals, as distinct from " transcrystalline " cracking, in which the individual crystals are broken through. Stress-corrosion cracks, when opened, show a granular texture which does not, however, sparkles as it does in true transcrystalline fractures. There is also usually some discolouration or other evidence of chemical attack. The occasional areas of sparkling crystallinity which are sometimes seen in such fractures, are usually indicative that some of the crystals have broken under the mechanical influence of the stress. The final break may be in the form of a true brittle fracture, or a ductile (fibrous) one, depending upon the properties of the material and other factors. The term " stress-corrosion cracking " should properly be applied only to the initial stage, before the final rupture occurs.

SHEAR FRACTURE : This type occurs by shearing on surfaces which are usually at about 45° to the principal tensile stress, or on surfaces of maximum shear stress in torsion. A familiar example is the 45° fracture which is commonly seen in tensile test specimens, and in the " walls " of the well known cup-and-cone fracture of cylindrical tensile specimens. The surface has a characteristically " silky " appearance, which is often interpreted as evidence of ductility.

" MAT " FRACTURE : This term denotes the dull, fibrous texture sometimes found in fracture surfaces which are perpendicular to the direction of the tensile stress, as or example the " bottom " of cup-and-cone fractures.

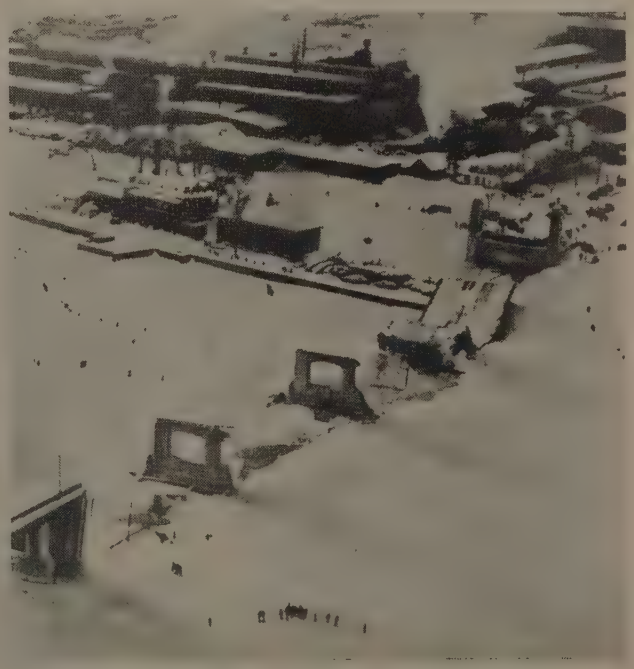


Fig. 5.—Failure of Duplessis Bridge

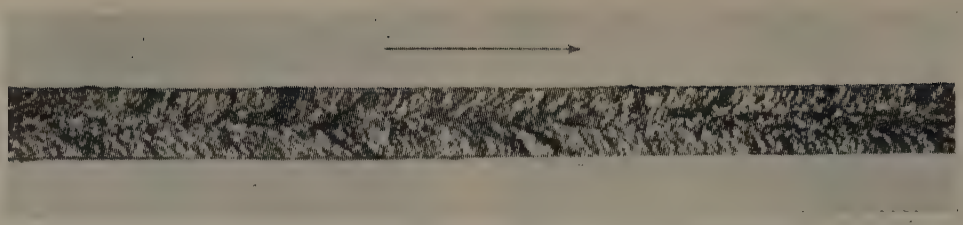


Fig. 6(a).—Fracture of Ship Plate 0.339 in. thick

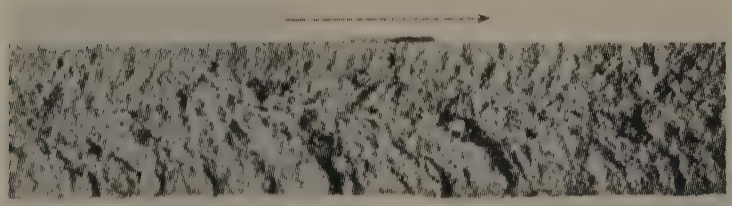


Fig. 6(b).—Fracture of Ship Plate 0.91 in. thick

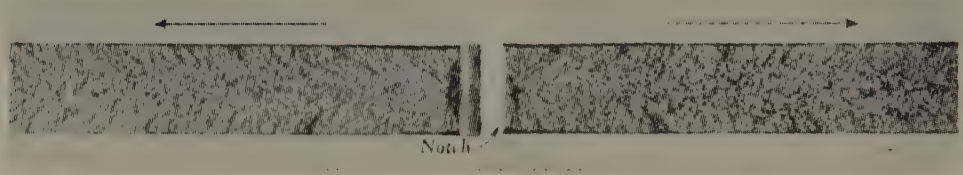


Fig. 6(c).—Fracture produced in laboratory, 0.532 in. thick[8]

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CRYSTALLINE FRACTURE : This texture is characterised by a sparkling appearance, denoting that the individual crystals have broken on planes determined by the regular arrangement of the atoms in the crystals. This process of fracturing on crystallographic planes is technically known as " cleavage," which term is sometimes applied to crystalline fractures. It is also sometimes applied, incorrectly, to brittle fractures generally.

The last three types of texture may, and often do, occur in conjunction. For example, a given fracture may contain portions of the shear texture, usually at the edges, and other portions, usually in the middle, where the mat and crystalline textures are intermixed.

BRITTLE FRACTURE : This term is intended to describe the kind of fractures with which we are mainly concerned here, and of which the main characteristics are as follow :

1. Brittle fractures progress at high speed, usually with considerable noise, and with little local plastic deformation, even when caused by static loading.
2. The fracture face is mainly plane, and perpendicular to the tensile stress.
3. The fracture faces are usually rough, and marked by ridges which lie roughly parallel to the direction of propagation, as distinct from the markings of fatigue fractures, which are perpendicular to the direction of propagation. In the case of a plate of uniform thickness, the markings take the form of a " chevron " or " herringbone " pattern, as shown in Fig. 6. These chevrons have the important property that they enable the origin of the fracture to be traced, since the apices of the chevrons point towards the origin. The reasons for the occurrence of this particular pattern have been discussed elsewhere,[8,11] and it need only be said here that the ridges forming the chevrons appear to be the tracks left by the advancing fracture front, which in plates of uniform thickness has a roughly parabolic shape. The chevrons also give some indication of the relative brittleness of the fracture, since they tend to diminish in clarity as the brittleness increases, and are practically invisible in totally crystalline fractures.
4. The faces of brittle fractures in mild steel usually show some crystallinity interspersed with the mat fibrous texture. The crystallinity increases with the brittleness, so that the texture may range from mainly fibrous, or mat, to fully crystalline, according to the relative brittleness.
5. Along the edges of the fracture, i.e. at the plate surfaces, there are " lips " of shear fracture, having a silky texture. These lips vary in width according to the brittleness, and are so small as to be practically invisible in fully brittle fractures.

These characteristics are summarised diagrammatically in Fig. 7, which also shows the various stages of development of a progressing brittle fracture.

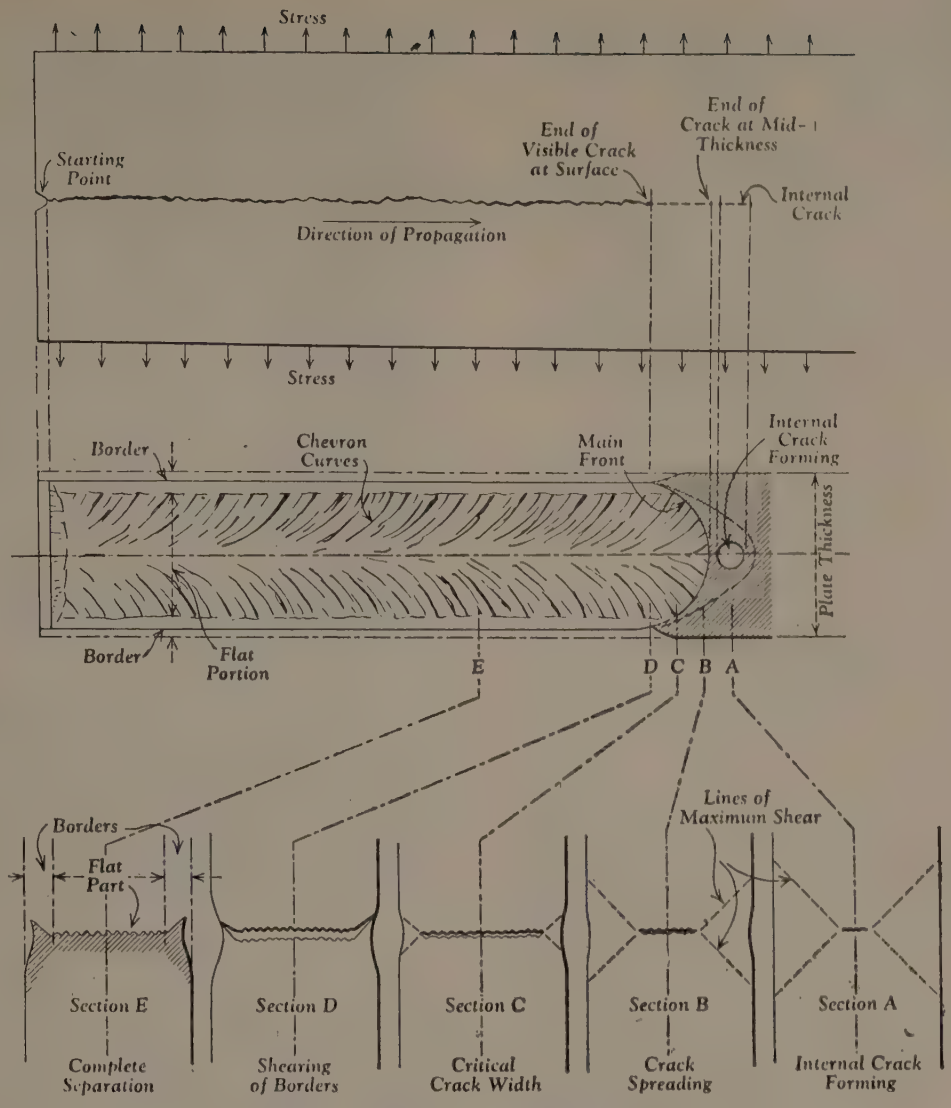

Fig. 7.—Features of Plate Fractures[8]

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EFFECTS OF BRITTLENESS ON STRENGTH

To most structural engineers,the strength of steel is a clearly understandable property, which is reflected in the ordinary tensile test. It is widely and implicitly assumed that steel will not fracture unless the stress exceeds the Ultimate Tensile Stress as revealed by the tensile test. This value is used as a basis for design, after the application of a factor of safety, the object of which is to allow for possible overloads which cannot be foreseen or estimated. Occasionally the factor of safety is increased to allow for fatigue or impact, and sometimes allowance is made for stress concentrations at discontinuities in the structure.

It is becoming increasingly apparent that these simple concepts, which have done yeoman service in the past, are inadequate to cope with the conditions of modern design, at least in cases where fatigue, stress-corrosion and brittle fracture must be considered. Dr. Weck has pointed out, in the paper referred to,[7] the futility of allowing for fatigue by simply increasing the ultimate strength, and the same applies to stress-corrosion and brittle fracture.

The strength and ductility of a steel structure depend on the condition, or state, of the material in the manner indicated diagrammatically in Fig. 8, which represents a typical load-deflection diagram for a structure of given form and material. In general, the behaviour up to the point of commencement of fracture is the same, irrespective of whether the eventual fracture is brittle or not. Thus, at a temperature at which the material is fully ductile, the shape of the diagram follows the line ABCD. At lower temperatures, when the material is very brittle, the fracture, which has commenced in a fibrous manner at point B, becomes brittle at some point C, after which the load drops to zero without further significant deformation. As the temperature is further reduced, the point C approaches B, and at very low temperatures brittle fracture may occur to the left of B, say at C^1. In extreme cases, the point C^1 may approach the yield point, but rarely, if ever, goes below it. Even in the most brittle condition, the specimen usually yields before fracture. This point has been brought out by A. A. Wells in a recent paper,[9] and the effect has been observed in specimens of all sizes, up to the size of normal structural members, even when severely notched.

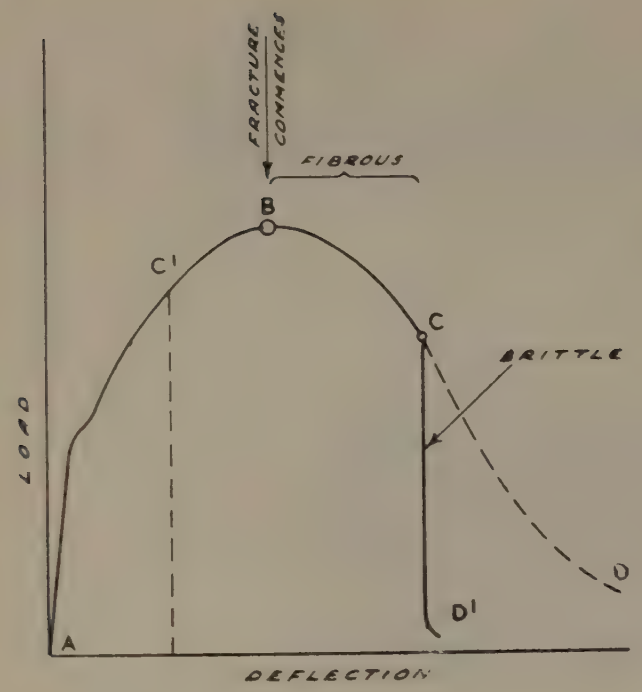

Fig. 8

If, however, a notched specimen is loaded to some stress, which may be lower than the yield point, and is then struck a sharp blow, brittle fracture may suddenly occur, or on the other hand it may not. Whether a fracture will occur or not depends on two factors, namely, the stress, and the toughness of the material.

This behaviour may, perhaps, be better understood by the following analogy. If a sheet of elastic material, for example rubber, is stretched between two parallel fixed bars, and then a nick is made in one edge, a fracture may or may not occur, depending upon the stress and the toughness of the material. The condition is that fracturing will only occur if the elastic energy released by its extension is greater than the work done, in the same extension, against the resistance offered by the material, which resistance is called the "toughness." Thus, a tightly stretched rubber sheet would probably fracture right across, but an equally stressed sheet of canvas, which is tougher, probably would not. Either material, however, would clearly have a minimum stress below which it would not fracture.

Robertson[10] has shown that steel behaves in just this way, and that the stress at which a brittle fracture can occur diminishes as the temperature is reduced. He prepared wide tensile specimens, notched as shown in Fig. 9(d), and then cooled them in such a way that the temperature varied across the width, the coolest part being at the edge that was notched. He then applied various stresses to the specimens, and artificially initiated cracks by striking with a pneumatic hammer, or bolt gun.

The results gave graphs of the kind shown in Fig. 10, in which the mean stress is plotted against the temperature at the point where the fracture stopped. It will be noted that the typical graph consists of two distinct branches, marked AB and BC in Fig. 10. The vertical branch, BC, indicates a temperature above which a brittle fracture cannot be produced at all in this particular steel, which temperature has been termed the "Upper Arrest Temperature." Above this temperature the resistance of the material to the propagation of a fracture is so high that the elastic energy which can be stored in the specimen is insufficient to extend the fracture.

The sloping part, AB, denotes the minimum mean stress required to propagate a brittle fracture in this material at various temperatures. This stress might be termed the "brittle strength" of the material, and it is clear that this may be considerably below the yield point as determined by the ordinary tensile test, and may even be below the appropriate working stress normally assigned to the material.

The increase in strength with increasing temperature is due to the corresponding increase in toughness, and at the point B the toughness reaches a level at which the elastic energy stored in the specimen is insufficient to overcome it.

Robertson has investigated a large number of steels, and has found that both the upper arrest temperature and the brittle strength vary considerably, but bear little or no relationship to the yield or ultimate stresses shown by the ordinary tensile test.

It is necessary to observe here that the account just given has been simplified for clarity. There appears to be a "thickness" effect which complicates the determination of the intrinsic "brittle strength" and "upper arrest temperature."

INITIATION OF BRITTLE FRACTURES

It has been seen that brittle fractures can progress at relatively low stress when they have been artificially initiated. It remains, however, to consider how such fractures may be initiated in actual structures.

The many investigations which have been made confirm that brittle fractures require a considerably higher

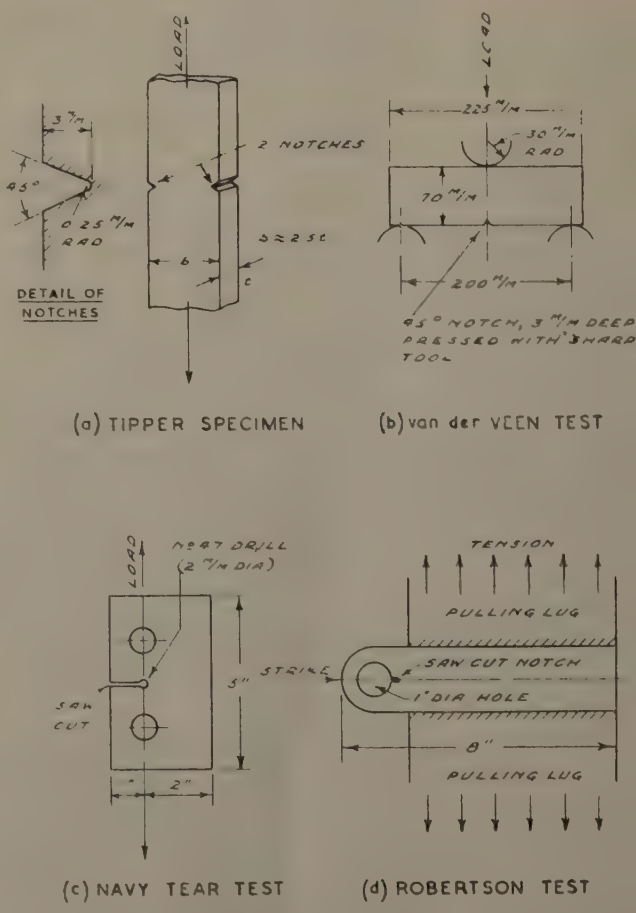

Fig. 9

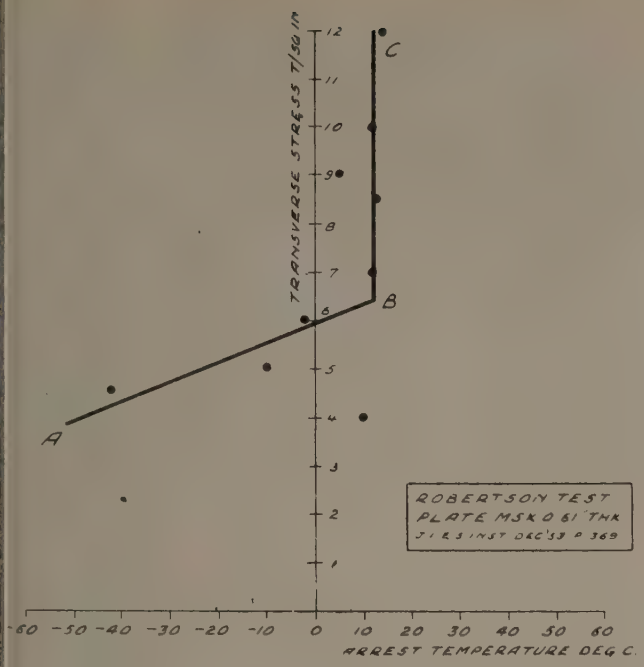

Fig. 10.—Typical indications of Robertson Test

average stress to initiate than to propagate them. This is a very fortunate fact, which has probably protected many structures from disaster in the past. It forms a kind of protective barrier, which, however, may be broken down by many circumstances, some of which are difficult to control. It appears that for fracture initiation to occur at low stresses, some kind of notch must be present, and this may take many forms, ranging from a design feature, such as a sharp reentrant corner, to a minor defect in a weld or a small fissure at a sheared edge or punched hole. Fatigue and stress corrosion can also, obviously, produce very dangerous sharp notches. In addition, it appears that the material at the root of the notch must be in a brittle condition, which may be caused by work-hardening or by metallurgical effects, such as strain-ageing or quench-ageing. Such effects are often present at the roots of notches in actual structures, owing to the processes by which the notches are produced. In particular, welds in which there are fissures, however small, may fulfil the conditions for initiation, since the material in the vicinity may have been work-hardened by the plastic flow which inevitably accompanies the thermal contractions, and it may also be strain-aged due to the heat cycle following the deposition of the weld. It may be for these reasons that fractures seem to be more frequently initiated in welded than in riveted structures, and it seems probable that the benefits which are known to accrue from the heat treatment known as "stress relieving" may be due to improvements in the metallurgical condition of the material in the vicinity of the welds.

Impact is also a potent factor in overcoming the initiation barrier, and if the other factors, such as stress, notch and metallurgical condition, are sufficiently severe, only a small shock may be required to start the fracture.

TESTS FOR NOTCH DUCTILITY

Many different forms of test have been suggested to indicate the temperature below which different steels may be liable to brittle fracture, given the appropriate conditions of stress etc., but at present there is no unanimity as to the most suitable test.

The available tests fall into two main groups, i.e.,

(a) Notch impact tests, which depend for their indications upon the measurement of the work done (or energy absorbed) in breaking a small specimen by impact. Examples of this group are the familiar Izod and Charpy tests, and their many variants. (Fig. 11.)

(b) Static notched tests, which depend mainly upon displaying the type of fracture which occurs when a standard notched specimen is subjected to a standard type of static loading, which may be either bending or tension, or combinations of these. Examples are the Tipper notch tensile test,[11] the Navy Tear Test,[12] the Van der Veen slow notched bend test,[13, 14] and several others (see Fig. 9).

All these tests should be carried out at various temperatures, in order to reveal the effect of temperature on the particular material.

The Robertson test, previously referred to, does not fit into either of the above categories, since it depends on the artificial initiation of fracture, by impact, in a notched specimen subjected to tension.

It is not possible within the scope of this paper to discuss fully the relative merits of these many forms of tests, but it may confidently be said that they all require expert interpretation. Moreover, there is no known simple method for correlating the results of one test with those of another, even when the apparent differences between them are slight.[15].

In general, however, it appears that if in any such test the fracture is predominantly crystalline, there is danger of brittle fracture of the tested material when built into a structure subjected to the same, or lower, temperatures and submitted to fairly high stress, provided that the conditions for initiation, outlined earlier, are fulfilled. For this reason it is important to consider the character of the fracture in such tests, in addition to the other indications, such as energy absorption.

In this connection it is interesting to observe that in a small notched impact test, such as the Charpy V-notch or Izod tests, a high energy absorption may be associated with a high percentage of crystallinity in the fracture. This is illustrated in Fig. 12, where the estimated percentages of crystallinity are plotted against energy absorption and silicon content for a large number of steels, all in the 26/32 t/sq. in. tensile range, tested at 0°C. by the Charpy V-notch impact test.

CORRELATION OF TESTS WITH SERVICE BEHAVIOUR

It is obvious that any satisfactory test should correlate with the service performance of the material tested. The correlation need not, of course, be direct, particularly if the object is merely to decide whether one material is "better" than another, without necessarily saying how much better. In other words, a test which would place several materials in the correct order of merit would be satisfactory in many applications. The difficulty, however, is to decide what is to be the criterion of merit, and this difficulty has not yet been satisfactorily resolved.

A great deal of work has been done with the object of establishing correlations between various tests and service performance. For example, the U.S. National Bureau of Standards[16] applied the Charpy V-notch test to a great many samples of steel, taken from fractured

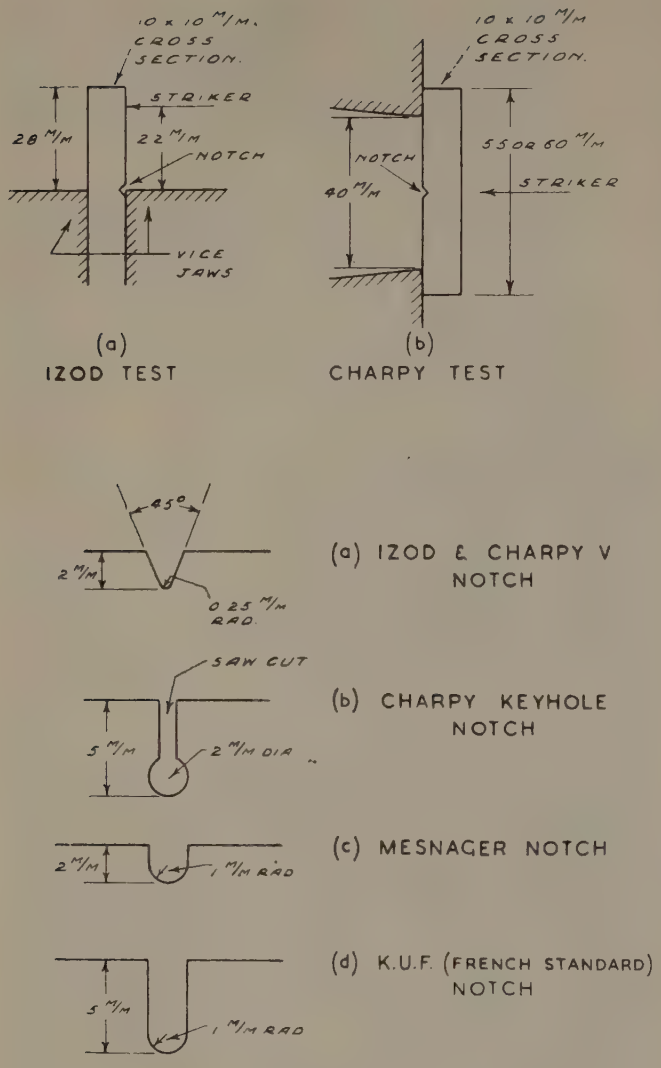

Fig. 11

ships, and found that if the temperature at which the energy value reached 15 ft. lbs. were taken as the criterion, the samples could be fairly well classified according to whether they contained the origins or ends of the service fractures, or whether they were traversed by the fractures. On this basis it was concluded that the " 15 ft. lbs. Transition Temperature " was a satisfactory criterion. This conclusion has, however, been criticised[17] on the grounds that while it applied to the particular type of steel involved in the ship fractures, it could not be used generally for all types of structural steel.

As regards the other group of tests, which rely on character of fracture as their main indication, the position is rather more satisfactory, since it has been found that a wide variety of such tests agree with each other and with large scale tests, in regard to the order in which they place steels. In particular, the Tipper notched tensile test (Fig. 9(a)) has shown full agreement with the types of fracture which have occurred in all the actual service failures to which the test has been applied.

The present position in regard to tests for notch ductility, however, cannot be regarded as fully satisfactory, and considerable further research is required. Such research is indeed in progress.

PRACTICAL MEASURES

It is seldom that engineers can await the ultimate outcome of research on problems of this kind. They must " get on with the job," taking such practical precautions as may be indicated by their experience and the currently available knowledge.

To deal with the present problem, several such precautions are clearly essential, namely the avoidance of all forms of notches and abrupt changes of section, the production of sound welds, and assurances regarding the notch ductility of the parent steel. The first two of these precautions are in the domain of the engineer and electrode manufacturer, but the last requires the full collaboration of steelmakers. This collaboration is readily forthcoming, and several special steels are already available, having a high degree of notch ductility. These special steels are, however, expensive, and it seems desirable that increased efforts

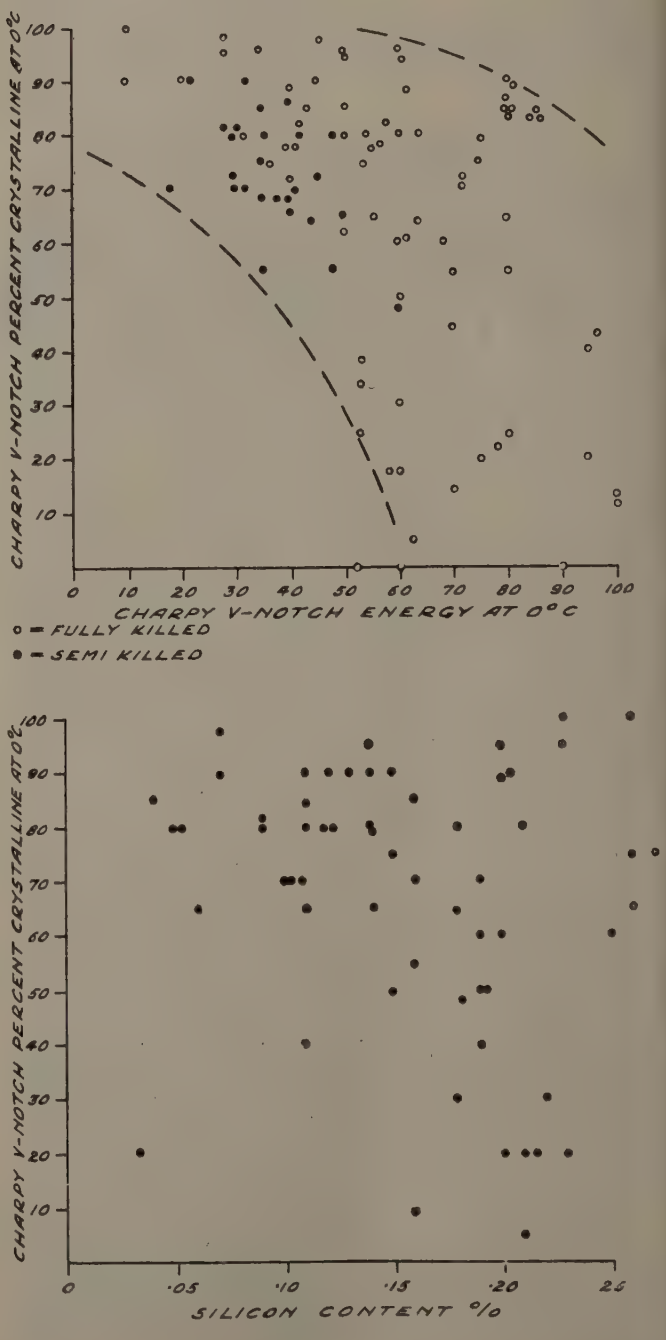

Fig. 12

should be devoted to the development of more economical steels having adequate notch ductility for the general run of welded structures, leaving the special steels for special applications.

Several "Utility" steel specifications have in fact been developed, such as those which are designed to comply with Lloyd's Rule P.403. This clause in Lloyd's Rules requires that steel for important parts of welded structures shall be specially approved by Lloyd's Register of Shipping for the purpose. Many different types of structural steel have been approved under this Rule, and these are now available in most countries. The implementation of this Rule depends primarily on expert interpretation of data, including tests for notch ductility, submitted by the various steelmakers, and subsequent maintenance of the approved specification under the supervision of Lloyd's Surveyors. Other authorities, such as the American Bureau of Shipping, and several others, have also introduced corresponding requirements.

In these circumstances it is now possible to obtain steels having fairly adequate notch ductility for most welded structures at reasonable cost, but further efforts are necessary not only to clarify the problem more completely, but to evolve more satisfactory steels at lower cost.

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Book Reviews

Statically Indeterminate Structures, by Chu-Kia Wang. (New York and London : McGraw-Hill, 1953.) 424 pp. 9 in. × 6 in. Price $7.50 (56/6d.).

Amongst the large number of recent publications on structural analysis, this book by Dr. Wang takes a useful place on account of the clear and straightforward manner in which the subject of statically indeterminate structures is presented and dealt with.

The book is illustrated by 352 neat diagrams and is divided into twelve chapters including a general introduction ; deflection of statically determinate beams, frames and trusses ; analysis of statically indeterminate beams, frames and trusses by the method of consistent deformation ; the three-moment equation ; the slope-deflection method ; the moment-distribution method ; the method of column analogy ; analysis of fixed arches ; secondary stresses in trusses with rigid joints ; composite structures.

In these chapters the standard methods are explained along with their uses and limitations, and alternative solutions are worked out to various given problems.

The author, who is Associate Professor of Civil Engineering in the University of Colorado, has produced an eminently readable book well suited to a student working alone as well as for classwork, there being no less than 106 examples worked out in mathematical detail in the text along with 178 exercises to which numerical answers are provided.

A. A. F.

Exécution du Beton Précontraint, by L. Bourgine. (Editions Eyrolles ; Paris, 1954.) 24 cm. × 16 cm., pp. 116. Price 1,350 F.

This well-produced and profusely illustrated account of modern pre-stressed concrete practice is intended to supplement the many theoretical dissertations already available. It is essentially a practical treatise with four chapters covering preparatory work, site organisation, constructional control and cost estimation. The text is very conveniently arranged in a series of short, numbered paragraphs each on a particular point of interest. The illustrations range from graphs of materials' properties, through items of special equipment, to full scale constructional work such as the placing of a heavy pre-stressing cable which takes some twenty men to handle. In the chapter on cost estimation actual prices which might have no more than local and transient significance are not discussed, but every factor likely to have a bearing on cost is outlined and its contribution estimated, either in the form of man-hours for a particular process or in the manner most appropriate to the item in consideration. One appendix lists ten instructional films on methods of pre-stressing and the other outlines the method of Faury and Caquot for estimating the optimum granular composition of the concrete. The book will be of undoubted interest to every structural engineer concerned with design or construction in pre-stressed concrete.

E. H. B.

Soil Mechanics Related to Building, by John H. G. King and Derek A. Creswell. (London : Pitman, 1954.) 168 plus xvi pp. 7½ in. × 5 in. Price 16s.

Most civil and structural engineers are keenly aware of the importance of proper soil studies preparatory to constructional work. It is surprising, however, that after some eighteen years of soil mechanics as a recognised science, there is very little appreciation of its value amongst those concerned with the general run of building work. A book written primarily for their benefit should therefore prove a most useful addition to soil mechanics literature.

The first chapter of this book provides, in a most readable and interesting form the essentials of geology which should be known by all who are concerned with building work of any consequence. The next chapter, on Site Investigations, describes very briefly nearly all that is required for most purposes, and should disillusion anyone who supposes that a proper site investigation is necessarily a very costly operation. The next four chapters deal successively with Testing, Classification, Bearing Capacity and Settlement, Earthworks and Retaining Walls, and cover nearly every problem likely to be met in building work. Finally, the last two chapters deal with Foundations, and in concise terms cover a very wide field. These chapters alone could with advantage be studied by anyone in the process of considering how best to ensure the safety of his building, with the maximum economy. Their study should also help in guarding against falling into the many common pitfalls connected with foundation work.

A criticism which might be made is the absence of any reference to mining subsidence. Admittedly, this subject is really outside the scope of soil mechanics ; it is, however, a problem which is confronting engineers and builders with increasing frequency. To ignore it when working in areas in which it may arise is courting trouble every bit as much as failing to make proper studies of the soil. It is considered that a book which is intended primarily as a guide to the practical man should include some remarks on mining subsidence and larger scale earth movements.

The authors are to be congratulated on managing to cram a large amount of information into a very small space. The book should prove valuable not only as a text book, but also as a medium for encouraging the closer study of soil as an engineering medium.

C. B. B.

Mechanics Part 2 : Dynamics, by J. L. Meriam. (London : Chapman & Hall, 1952.) 9 in. × 5½ in. 331 pp. Price 32s.

This volume by the associate professor of engineering design in the University of California is the second part of a work of mechanics of which the first part, published separately, dealt with statics. It presents the basic principles of dynamics and illustrates them by numerous fully worked out examples, which are supplemented by over 600 other examples, mainly with answers, on which the reader may test his own ability to apply the principles given in the text. In selecting these examples which, by the way, are carefully graded in order of difficulty, the author has, where possible, chosen practical problems involving real situations as well as idealised ones and it is fair to say that they cover a refreshingly wide field. It should, however, be noted that the textbook deals only with rigid body dynamics and not at all with the dynamics of non-rigid bodies, e.g., the response of structures to impact loading.

The scope of the text is indicated by the chapter headings, which are :

Principles of Mechanics ; Kinematics ; Principles of Kinetics ; Force, Mass and Acceleration ; Work and Energy ; Impulse and Momentum ; Periodic Motion ; Moments of Inertia ; Useful Tables.

The author opens with an interesting brief historical survey of the subject. He assumes that the reader has a knowledge of the calculus, but wisely considers it desirable to give a note on the accuracy of working (in terms of significant figures).

The whole book is very fully illustrated with clear diagrams and a useful typographical aid is provided in that all important formulae are shown in bold. It can be confidently recommended as a useful addition to the textbook literature of the subject.

L. A. B.

Beam Formulas, by A. Kleinlogel, translated by H. G. Lorsch. (London : Crosby, Lockwood, 1953.) 143 pp. 9½ in. × 6 in. Price 40s.

Dr. Kleinlogel has become well known as the author of several publications, and this volume is in the main a translation of the seventh edition of his " Belastungsglieder." As the title implies, it is a collection of formulae for calculation of data required for the design of beams, that is reactions, shearing forces and bending moments.

Formulae for slopes at the ends, and deflections of the simply supported beam with constant cross section are also given.

More than seventy conditions of loading of the simply supported beam, including temperature variations and cases of moments applied at the ends, are dealt with.

Ready formulae for the fixed beam and the propped cantilever are given for ten frequently occurring conditions of loading.

Continuous beams are not specifically dealt with except in so far that when the moments at the supports are known, shearing forces and bending moments in any span may be derived from the formulae for the simply supported beam. The procedure then is to add algebraically the results due to the load and those due to the moments at the supports, applied as external moments at the ends of the span. As to the moments at the supports, essential for the application of the formula, they must be determined first, by any method preferred.

In addition to the formulae, there are many tables of coefficients to facilitate the calculation of results for more important cases of loading, and clear illustrations showing the forms of the shearing force and bending moment diagrams accompany the formulae for all cases of loading on a simply supported beam.

Preceding the formulae is a brief instruction on their use. The manner in which the formulae are derived is explained in an Appendix. A second Appendix gives graphs enabling moments of inertia and centroidal distances for T-beams to be determined rapidly ; these should be useful to anyone designing indeterminate structures.

With a few exceptions the notation is that commonly used in this country. The matter is well set out, and the book should prove a time-saver to those with sufficient experience to use the data with discretion.

There is one small point that may be confusing to the less experienced. The symbols used throughout to denote slopes of the elastic line at the ends of a beam, have also been used for the slopes of the bending moment curve at these points. (Appendix I, Fig. 4.)

B. E.

(*Book Reviews continued on page* 34)

Load Distribution in Prestressed Concrete Bridge Systems*

Discussion on Paper by Dr. P. B. Morice, B.Sc., and Mr. G. Little, M.Sc.

THE President introduced the Authors, who then presented their paper.

Discussion

THE PRESIDENT proposed a hearty vote of thanks to the authors for their paper and for their extremely interesting presentation of it.

(The vote of thanks was warmly accorded.)

MR. C. S. CHETTOE (Ministry of Transport) said that in the years before the war the form of the Ministry of Transport Standard Loading, which always provided in some degree for the passage of abnormal loads, might have tended to obscure the importance of transverse distribution of single loads on bridge decks, since the loading consisted of a uniform and a knife edge load extending across the whole width of the deck.

Since the war there had been a great increase in the number and weight of abnormal vehicles, and the Ministry had now a special single abnormal design loading ; that had been illustrated by the Authors. It was therefore most important that we should be able to calculate the distribution of heavy single loads across bridge decks with reasonable accuracy. That applied not only to the design of new bridges, but to the assessment of the strength of old ones. The movement of abnormal loads was controlled and we could, if necessary, direct such loads to pass over the strongest part of a bridge deck, which would usually be the longitudinal axis.

The Authors had, so far as British engineers were concerned, made a large contribution to engineering knowledge by making the theories of Messrs. Guyon and Massonnet known and readily understood in this country. They had also contributed quite extensively to those theories and had made them convincing, in so far as practical application was concerned, by their numerous tests on models which were sufficiently large to represent reasonably full-scale conditions.

The theories were not and should not be limited to prestressed concrete in application. With suitable modifications to the value of E, etc., they should be as readily applicable to other structural materials.

A great merit of the Authors' work was the way the application of the " no torsion " analysis of Guyon and the " torsion " analysis of Massonnet had been dovetailed together so that the application of the latter was a simple expression of the work of the former. The combination of the two methods had been presented in a practical form, and used quantities familiar to and readily estimated by the practising engineer, so that application in a design office was reasonably straightforward.

In using those methods, engineers should not forget that they were approaching very closely to an exact estimation of the apportionment of the load to various members. Consequently it was essential to ensure that what they built was what they had assumed. To do so they should make sure that the design assumptions they had made were fully realised ; and, for instance, having taken a value for a moment of inertia in a cross member, they must see to it that that was fully realised at all conditions of loading, i.e., they must design the cross members and connections so that stresses in them would not open cracks or bring about local failures which would alter the stiffness properties. Mr. Little had referred to that and had illustrated the bridge test, which brought the point out.

It was therefore of considerable value that the Authors, having found some discrepancy between the behaviour of models and the theory, in that respect had given us a lead, in their introduction, to a satisfactory way of calculating the transverse members.

MR. A. GOLDSTEIN (Associate-Member) added his congratulations to the Authors on their clear exposition of a difficult problem, one which had exercised the ingenuity and judgment of engineers for a long time. He agreed entirely with Mr. Chettoe that engineers' judgment as against theoretically " exact " design had been vindicated in many cases, and some of the following would perhaps show the reason why.

Coming to a consideration of the historical aspect, and leaving aside purely empirical methods which had been developed over the years, Mr. Goldstein said that obviously one way in which a bridge could be designed would be to treat each beam completely independently, this being one limiting condition of the transverse problem ; diagrammatically this way was represented in Fig. 1a.

The second method, which had been used quite frequently in the past, was an attempt to produce a more consistent attack on the problem. This method assumes infinite transverse stiffness—the other limiting condition—and Massonnet attributes it to Engesser (1889). Since it assumes infinite transverse stiffness the method entailed an analysis rather similar to the analysis of eccentric rivet groups and could be represented diagrammatically as in Fig. 1b. Transverse moments and shears in the diaphragms could also be ascertained and it was shown that the secondary moments produced by point loads between nodal points were, in practice, negligible. Thus this method presented a great step forward. Massonnet, using Engesser's method, had investigated some existing bridges (not originally designed as properly interconnected) and had found the difference between maximum moments in the different beams to be only 10 per cent. as against 19 per cent. given by the ordinary methods of calculation. Thus if a bridge were proportioned and designed as interconnected considerable

*Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1 on Thursday, March 25th, 1954. Lt.-Colonel R. F. Galbraith, M.C., B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E. (President) in the Chair. Published in " The Structural Engineer," Vol. XXXII, No. 3, pp. 83-111, March 1954.

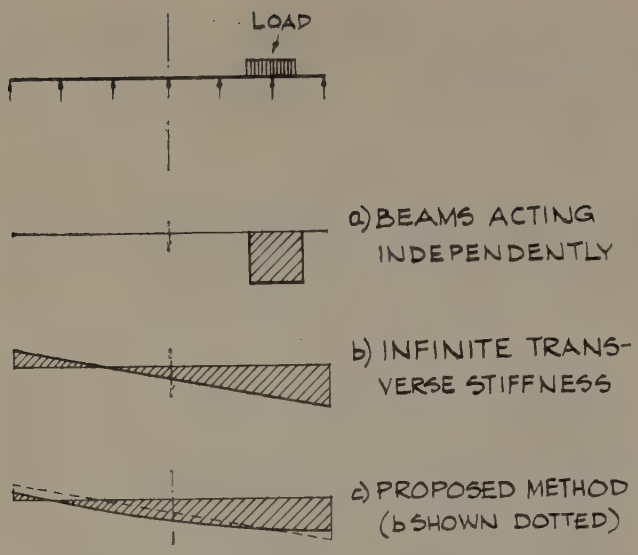

Fig. 1.—Diagrammatic representation of three types of transverse load distribution

economy would result. The astonishing thing about this Engesser method was that, notwithstanding its doubtful assumption, it gave reasonable and applicable results in a fair proportion of practical cases. Massonnet gave its region of applicability as :

$$\delta = \frac{b}{2a}\sqrt[4]{\left\{\frac{a}{b}\cdot\frac{E\,I \text{ beams total}}{E\,I \text{ diaphragms total}}\right\}} > 0.3$$

which corresponds to θ as follows :

$\frac{L}{b}$	$\theta <$
6	0.47
5	0.45
4	0.42
3	0.39
2	0.36
1	0.30

Those values of θ covered, in Mr. Goldstein's view, a fair proportion of practical cases and within the region of application Massonnet considers this method of " equal importance to ordinary methods giving results which are both more correct and more advantageous."

However, of recent years, methods of construction had developed which lent themselves to a more efficient distribution of load carrying materials on the one hand, but bridges had to carry ever-increasing abnormally heavy point loads—difficult to distribute by earlier methods—on the other. It was appropriate therefore that methods of design and calculation should develop in parallel. The method described by the Authors was such a development, giving both the earlier work of Guyon and the later extensions and amplifications by Massonnet. This method could be presented diagrammatically as shown in Fig. 1c and the resulting load distribution curve would seem to be a very reasonable one.

Referring to a few detailed points in the Paper :

There was the question of the optimum value of θ.

This, of course, depended on what was made the criterion. For example, Massonnet considered the value of 0.669 better than 1.0 as it gave reduced moments in the ratio of 1.28 to 1.37. However, as he had pointed out, the optimum value of θ, regardless of technical considerations, was that value which gave the cheapest bridge and this could be obtained by a few trials and errors and subsequent comparisons of the resulting designs with detailed prices.

On the question of applicability it was worthy of note that Massonnet considered that " one makes large enough (assez forte) errors in applying the proposed method to bridges of very small spans (e.g. less than 8 metres), because the dangerous load of these bridges is obtained from a single load situated in the middle transverse section, which differs strongly from the sinusoidal load considered for the analysis of transverse distribution." He went on to say that for these small bridges the empirical method obtained from the researches of the University of Illinois could be used without great loss of economy. However, considering the experimental results in the Paper it seemed that Massonnet might have been unduly pessimistic, and in any case more load terms of the Fourier series could have been taken into account to produce greater accuracy.

On the question of load terms could the Authors say how many terms they had taken when comparing theoretical with experimental results. The agreement in this comparison was certainly very satisfying.

In the paper, the transverse distribution and the bending moments in the beams was treated in some detail, the bending moments in the diaphragms in less detail but nothing was said about shear. The expression for shear was similar to the expression for My on p. 95 except that a different constant τ is used instead of μ. The form of τ was given in the original papers but not any tabulated values. Whilst of course one could obtain shear values from the plotted bending moment diagrams, perhaps the Authors could, for completeness, elaborate in the written discussion both on the question of shear and on bending in the diaphragms.

When considering the factor μ, it was, of course, a very long job to check all the values arithmetically, but as a first stage Mr. Goldstein has used plotting and integration methods for a check on those values which he had had to use so far. Fig. 2 shows the influence lines for μ, i.e. bending moment, for $\theta = .669$ and $\alpha = 0$. The area of each curve should be zero, and in one or two cases adjustments had to be made. The main error, however, occurred at point $\frac{b}{2}$ when the load was at $-\frac{b}{4}$, where instead of the value of 470 the correct value appeared to be about 20. This, he thought, was just an arithmetical mistake. It was also interesting to note that for the lower transverse stiffnesses there was a sort of damped harmonic influence, which he gathered had been verified experimentally.

Mr. Little had spoken about the increased transverse moments. Mr. Goldstein had understood him to say that this applied to instances where α was nearly 1, i.e. slabs rather than grillages. In the case of the grillage, where the torsional stiffness was as a rule very small, he did not think that the transverse moment would be increased to the extent mentioned.

Finally, Mr. Goldstein put the following points to the Authors so that they could be elaborated in the discussion.

Frequently, a useful and economical arrangement for diaphragms in deep beams was to split them up into a top boom—integral with the slab and a bottom boom, this arrangement making with the beam stiffeners a

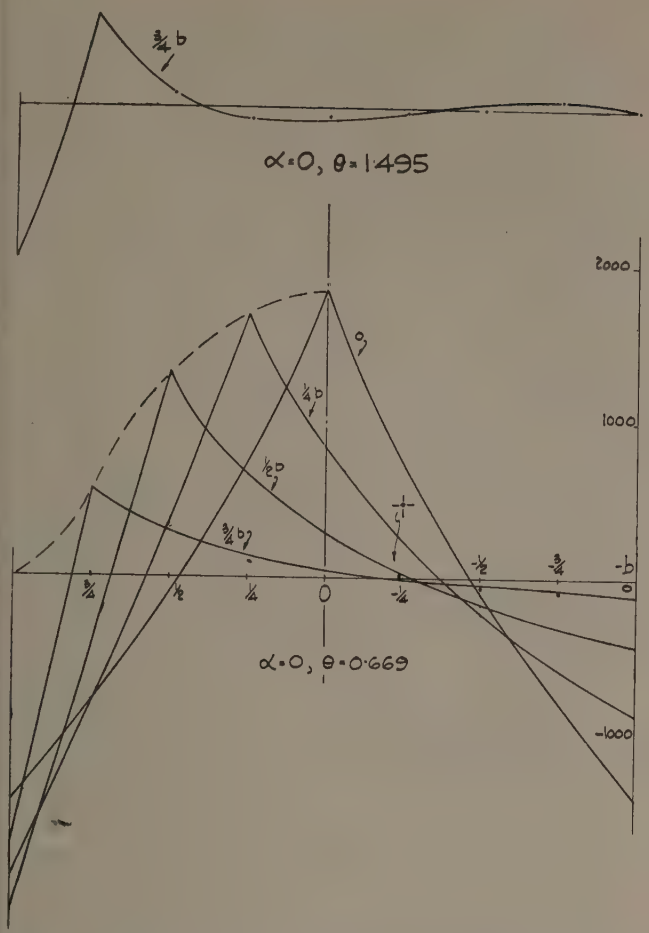

Fig. 2.—Influence lines for transverse bending moment coefficients

sort of Vierendeel type diaphragm. The problem was what to take as the effective inertia or stiffness for the diaphragm.

Another point was the effect of the slab on the transverse stiffness. Engineers would be most interested to obtain the results of the research that was being carried out. It would seem that some Tee effect of the slab on the diaphragm would have to be taken into account.

Engineers would also look forward to hearing about the results of experiments on the effect of skew. The University of Illinois had done a fair amount of work on this subject. He had been under the impression, from their report, that up to 30 degrees of skew the effects were small and generally negligible. Massonnet also would appear to agree with this view. For large degrees of skew however some further research would quite definitely be necessary.

In conclusion, Mr. Goldstein pointed out that reference 7 of the paper, i.e. a paper by Massonnet published by the I.A.B.S.E. seemed to be an abridged version of Massonnet's full paper published in the memoirs of "Travaux Publiques de la Belgique."[1] This was a very full article giving a host of useful information including a résumé of the Illinois tests. Unfortunately it was in French and perhaps the excellent translating service of the Cement and Concrete Association could be called on to issue a translation of this work as they had done so ably for other works.

Mr. A. J. Harris said the work described in the paper had all the virtues. It related to a problem which was real and actual, the correct solution of which promised economic advantage. The work had been carried out admirably from the practical point of view, the mathematics was sound, so far as one could see, and the Authors had been charitable enough to present it in a form in which the ordinary designer could use it. Finally, wonder of wonders, the practical results had agreed with those predicted; that seemed to sum up to a very considerable achievement.

On the point that the problem had not arisen in terms of reinforced concrete, he said the Authors had suggested that that was due to lower flexibility of reinforced concrete. He questioned that; he held that prestressed concrete had greater stiffness than reinforced concrete, and that the reason why transverse beams were rarely thought of in terms of reinforced concrete was that they had to be cast *in situ*, which complicated the form work considerably. Also the elastic properties of reinforced concrete were irregular and non-uniform; we did not obtain the beautiful straight lines such as those in the graphs in the paper.

Mention had been made of the effects of the value of prestress on the transverse rigidity. In point of fact, said Mr. Harris, the results showed that with higher values of prestress the diaphragms were stiffer. That seemed to raise a question. He asked if the Authors attributed that extra stiffness to the presence of more steel in the transverse section or to a modification of the modulus of elasticity of the concrete by applying a higher initial stress to it.

On the question of skew, he said that a practical method of design which was used quite a lot in skew bridges was to put in the transverse sections parallel to the abutments, and therefore at an angle to the beams, and to assume that they acted in the same way as a square grillage. He wondered what would be the Authors' comment on the validity of that assumption.

The simply supported grill described in the paper had no diaphragm at the supports. Clearly, however, a real bridge would have a substantial diaphragm here.

The question of the value of the transverse prestress was obviously vital. He pointed out that if the calculations were to give a true representation of what was happening, that transverse prestress must be central to the diaphragm, for otherwise a hog would be introduced into the diaphragm and there would be at the start a complex distribution of loading between the beams. It was possible that an uneven distribution could be turned to good effect; in certain circumstances, where there was a more heavily loaded beam in the middle of the bridge, the engineer might be able to relieve the load by inducing a transverse hog, but this can be done deliberately after appropriate calculation.

It was also true that thought must be given to the magnitude of the average prestress in the diaphragm relative to that in the deck. This is a function of the order of stressing; if the diaphragm is cast first and stressed and then the deck cast and stressed, there is a danger of much of the deck prestress forces being supported by the diaphragms thus understressing the former and overstressing the latter. Here, as elsewhere in prestressing, design must take account of constructional sequence.

Mr. R. E. Rowe (Cement and Concrete Association) described some work which was a continuation of that given in the paper and mentioned by Mr. Little in his introductory remarks.

Since no actual slab bridge had been tested, he said, it was thought desirable to test such a specimen to see if the theories were valid for the extreme case of full torsional rigidity, in other words, a structure where $\alpha = 1$. The slab constructed had a span of 57.8 in. and a width of 46.25 in., giving a θ value of 0.4. The

thickness of the slab was 1 in., so that the depth-span ratio and θ were typical of actual bridge structures. (A slide was shown giving a general view of the slab and the test equipment.)

It was found that for deflections the distribution factors given by Massonnet were accurate within 2—3 per cent. for loads between $\pm b/4$, but the discrepancy at the edge remote from the load increased for loads of greater eccentricity, being of the order of 9 per cent. in the extreme case, as was found in the tested grillages.

As the load was moved along the span the distribution remained appreciably the same on the transverse section under the load as for the central transverse section when loaded. The moments showed a greater discrepancy, as would be expected, being of the order of 20 per cent. Although Guyon[2] had pointed out that the assumption of a zero value for Poisson's Ratio necessitated an increase of between 10 and 15 per cent. in the theoretical stresses to arrive at the actual, the error in this case was felt to be due to the large strain gauge length used on the slab. For that reason E.R.S. foil gauges, of 1 in. gauge length, were used in further tests.

(A slide was shown of the general set up of active and dummy gauges and the recording apparatus.)

With that method of measuring strains, reliable readings were obtained. The main object of the tests carried out was to obtain data about the transverse moments in the slab for various positions of the load. Since Massonnet's analysis was based on the assumption that Poisson's Ratio was zero, the experimental μ values were calculated on that basis. The theoretical and experimental curves for the distribution of μ were shown in the third slide. (Fig. 3.)

The form of both sets of curves was practically identical, though the discrepancy in actual maximum values was quite large, falling between 18 per cent. for a load at 0 and 27 per cent. for a $b/2$. The cube roots of the μ values were plotted, as it was impossible to cover the wide variation in the distribution in any other way.

If a value of Poisson's Ratio was assumed, the magnitude of the μ values was altered, but the shape of the curve, in general, remained the same.

It was interesting to note that the ratio of the transverse to longitudinal moments was considerably higher than that normally allowed for. According to the theoretical analysis, the worst condition was for a central load on the bridge, the ratio being then 0.39 for a zero value of Poisson's Ratio. Using that assumption, the experimental ratio was found to be 0.41, but with a value of $\sigma = 0.15$, a normally assumed figure for prestressed concrete, that ratio rose to 0.53. From the curve of the distribution of μ it might be seen that that figure held for the region $-b/2$ to $+b/2$, in other words the central half of the slab, since it fell to only 0.48 at those points.

Similarly, for a load at b/2 the theoretical and experimental ratios were 0.351 and 0.354 respectively for $\sigma = 0$. So that it appeared that for the ratio of $\frac{My}{Mx}$ the theory gave good agreement with the experimental results, and it only remained to find a correction to be applied to allow for the Poisson's Ratio effect. The best method of doing that appeared, at the present moment, to be to increase the longitudinal theoretical moments by 15 per cent. to give the actual, in accordance with Guyon, and to take 50 per cent. of those for the transverse moment and design on that basis.

The effect of a number of loads along the centre line of the bridge was to decrease the ratio of the transverse to longitudinal moment at the centre. For instance,

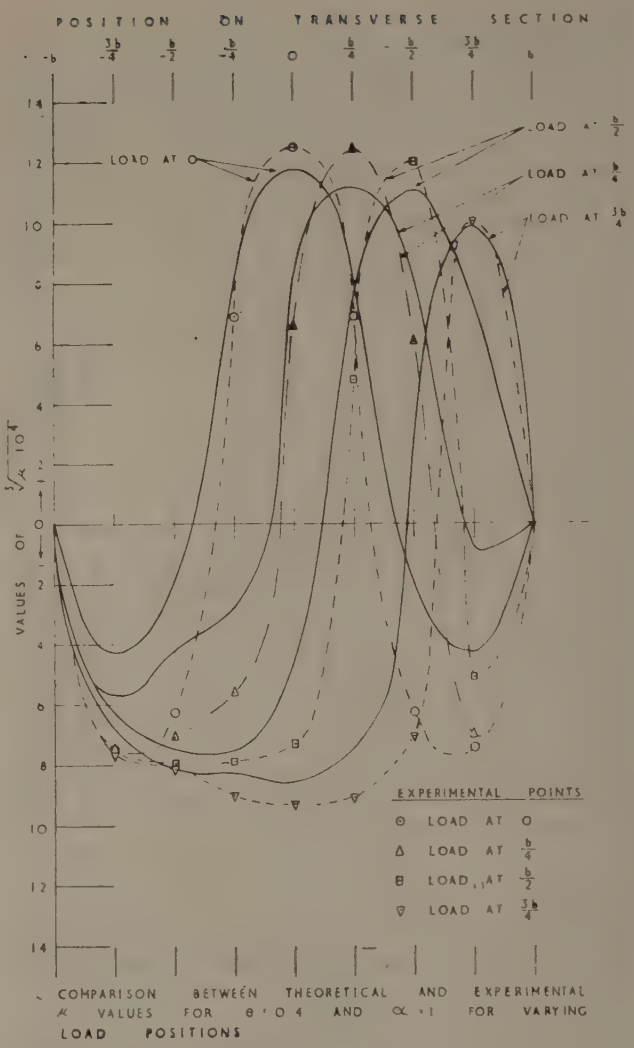

Fig. 3.—Comparison between theoretical and experimental μ values for $\theta = 0.4$ and $\alpha = 1$

from experimental readings and applying superposition, the ratio for equal loads at $\pm \frac{a}{2} \pm \frac{3a}{2}$ was 38.3 per cent., as against 53 per cent. for a single concentrated load at the centre.

As the load was moved along the span, the ratio of transverse moment to the longitudinal moment at the centre of the slab for a central load was :

Load at centre of transverse section	0	a/4	a/2	3a/4
$\frac{My}{Mx}$	53%	25.5%	13.4%	7.4%

Mr. Rowe added that further work was in progress to increase the amount of information available about the transverse moments and their distribution, and slabs with θ values of 0.5 and 0.6 were to be tested.

Mr. Wallace A. Evans (Member), after congratulating the Authors of the paper on a very fine piece of work, presented the result of some tests on an actual bridge, which came out very much in agreement with the results the Authors had obtained theoretically.

There were two spans, of 65 ft. and 75 ft. respectively, and there was 5 beams. There was a precast member outside which was used for prestressing the stiffeners. In addition there were stiffeners at the abutments and

on the end of each beam. The beams were not continuous, but were simple spans, and there were cross members between the 5 beams.

The point was made by Mr. Harris about putting the prestress in the slab was appreciated ; the stiffeners did not actually reach the slab.

The stressing of the bridge was rather interesting, in that it was done in two parts, first to carry the dead weight of the beams themselves and the live load of the decking, then the beams were placed in position and the second prestressing was applied, which took care of the impact load and utilised the advantage of the dead load.

The stressing of the diaphragm was intermediate between the first and second prestressings. That gave an interesting result, in that the actual application of the second prestress also showed the same kind of distribution of loading as that described in the paper, i.e., the prestress on one beam, causing the upward camber of that particular beam, also affected all the other beams, showing that there was a cross distribution of loading.

In the case where there were 5 beams, when beam No. 5 was being prestressed there was an upward deflection of 0.048 in., and the prestressing of that beam caused an upward deflection of the other beams. The deflection figures for the 5 beams were :

0.048 in., 0.037 in., 0.026 in., 0.028 in., 0.019 in.

That was very similar to the sort of line obtained from the graphs in the paper.

The results of an actual loading test tended to bear out the results given in the paper.

Defining the 4 spaces between the 5 beams as A, B, C and D, Mr. Evans said the load was applied on only one of those spaces at a time, and the main results were as follows :

When the load was on D, the deflections were :

	1	1½	7	17
When on B :	5	10	11	5
When on A :	9	17½	6	2

That, he concluded, confirmed to a very considerable extent the results obtained experimentally by the Authors of the paper. He intimated that he could give more figures in writing.

Mr. W. Henderson (Ministry of Transport) said the Authors had presented an interesting and particularly useful paper on a subject which was attracting a considerable amount of attention.

In carrying out tests on old bridges of doubtful capacity for the purpose of assessing their strength, he had continually been made aware of the existence of the phenomenon which they had reduced to a quantitative study. One had tried to take it into consideration empirically. It was satisfactory to be able to use that quantitative study, which came so extraordinarily close to the experimental results.

From the point of view of designers and of those who had to assess the strength of bridges, the methods by which the Authors had worked were especially interesting for two reasons : they had two very considerable advantages.

In the first place, the necessary calculations could be set out in tables as a piece of systematised arithmetic which, with the assistance of the curves, could be quite rapidly completed, and which provided its own checks at important points. He had recently completed a standard chart, based on the Authors' work, for that purpose, and it was fair to say, as a measure of the laboriousness involved, that it probably required less time and effort to handle than did the familiar arch chart. In other words, it did not involve a lot of work in relation to the results obtained.

The second advantage, which he considered even more considerable was that the person applying the method could see what was happening in the structure and could visualise the effect of increasing or decreasing the stiffness of the transverse members. That was an advantage which was not common to several methods which had been developed during recent years. The points made by the Authors in the discussion concerning the amount of transverse moment developed were extremely important, for it was essential that the members which were responsible for causing the redistribution should be strong enough to sustain the moments induced in them. The designer's primary concern would be with the main longitudinal beams, and the extent to which he could reduce the moments governing the design of them. Since the redistribution effects were based on dimensional properties, he might tend to forget about the bending stresses in the cross members. If, however, the transverse members were overstressed and cracked in bending under load, they would cease to have the dimensional properties assumed. Consequently the redistribution of load assumed would not be available. So that in using those methods one must be extremely careful to see that the transverse member would do under all conditions of loading what one assumed it would do in the calculations, particularly having regard to the loads that were travelling about the country.

The solution presented by the Authors dealt essentially with decks where each of the main longitudinal beams had the same second moment of area. In decks where a considerable eccentricity was possible for the live load, it might be that greater strength, and consequently greater stiffness, was required for the first or second beam from the edge of the deck. Similarly, when investigating the strength of existing bridges designed by orthodox methods, it was frequently found that the edge beam was much lighter than the interior beams. He asked if the Authors could suggest a way of dealing with the case where " I " for the edge beams was different from " I " for the interior beams.

In a system of calculation which had been used extensively in Germany since about 1938, devised by Dr. Leonhardt, there was provision for that, and it could be extremely useful. Unfortunately, Leonhardt's method did not consider torsion effects, and was much more laborious to use than were the Guyon and Massonnet methods. Those who knew it would recollect that the approach used was to assume a single equivalent cross member at mid span. Thereafter a series of equations must be solved, varying according to the number of actual beams. The solution of those equations gave influence line ordinates for distribution coefficients from which longitudinal and transverse moments could be derived.

The Authors had, understandably, made their study from the point of view of prestressed concrete decks. Mr. Henderson added to Mr. Chettoe's comment that there seemed no reason to limit it to those, and he suggested that the basic equations be stated in more general terms, so that longitudinal systems and transverse systems consisting of different materials were covered. That would allow for the case of steel beams with concrete deck slabs, a type of construction where the application of the theories would be of considerable value. It meant that in long span bridges the stringers could quite likely be reduced in section, and the dead load reduced a bit.

Mr. A. D. HOLLAND, touching on the point brought out by the Authors regarding the variation in the transverse prestress in the cross beams, said they had shown that there was a critical range for this, below which the cross members failed to develop their required stiffening effect or torsional restraint ; above that range secondary effects came in and, as they had said, there was in fact a critical value for the transverse prestress. Variations in the transverse flexural rigidity of the deck, and hence in the θ values, could have quite a significant effect on the K value on which the design had been based. Extra care from the point of view of tolerances in supervised workmanship might be called for to ensure that for a precision design such as we were now approaching with these methods the actual structure was in fact built as the designer intended it to be built. For example, a small variation of the axis of the transverse cables from the axis which the designer intended them to follow might lead to a significant departure from the design assumptions.

In connection with the ratio of transverse moment to longitudinal moment, when introducing the paper the Authors had mentioned figures of 50 and 55 per cent. That was a high proportion in relation to figures which had been used elsewhere. At the same time, this figure related to a longitudinal moment, which took transverse distribution fully into account. Many reinforced concrete slab deck bridges in this country which have behaved satisfactorily, have considerably less than 50 per cent. of the longitudinal reinforcement in the transverse direction. Possibly these decks were over-stiff in the longitudinal direction, and were not developing high transverse moments. In the case of the slab decks, would the reinfinement in design now put forward, which by taking the whole of the deck into consideration lead to a small saving in construction depth, achieve overall economy? Offsetting the saving in depth, there were increased transverse moments to provide for, and, as had been mentioned, the most economic design was the one for which provision in both the longitudinal and transverse directions was a minimum.

Referring to the case the Authors had quoted, in which a full scale test was made on a bridge, he asked if they would state the relation between longitudinal moment as measured and the design moment under the same loading.

Finally, he said that the transverse moment was being related to a longitudinal moment. He put it to the Authors that it would seem more logical to relate it to the difference between the maximum and minimum longitudinal moments across the deck.

Written Discussion

Mr. J. E. SPINDEL : the Authors are to be complimented not only for confirming experimentally that Lagrange's equation can be used to calculate the distribution of load in grillage type bridge decks, but also for the consideration they have shown in publishing charts which obviate the need for the intimidating and repulsive calculations usually associated with that equation.

It is interesting to note that the differential equation obtained by substituting the Levy series in equation 1 is of the same form as that giving the deflection of a beam on elastic supports subject to an axial tension as well as bending. This analogy is useful because for transversely stiff decks (small values of θ) the calculation is reduced to the approximation mentioned by Mr. Goldstein when $\alpha = 0$. Further, a very comprehensive collection of formulae[3] is available for the solution of the beam problem which covers cases not solved by the Authors' chart, such as the effect of moments applied at the edge of a slab—say by transverse eccentric prestress. Such prestress, incidentally, can be used to transfer load from the edges of a slab to the centre.

A point on which further information seems required is the effect of higher harmonics in the Levy series. Since the value of θ is proportional to m, where m is the coefficient of the term in the series, the distribution in infinite and semi-infinite slabs or deck systems is inversely proportional to m so that neglect of say the higher harmonics representing a point load leads to an overestimate of the amount of distribution. The error in the calculated deflection, being proportional to m^{-3}, is small, but the discrepancies in bending moments are more serious since the latter are proportional to m^{-1}. There seems to be some indication of this in recently published results of tests on joist-in-concrete slabs.[4]

This effect of harmonics has led to the conclusion that the distribution of a point load decreases as the load approaches the abutments and therefore influences the variation of bending moment for which longitudinal beams have to be designed. Its most serious aspect as far as the scope of the paper is concerned is the fact that the relative importance of harmonics depends on the position and distribution of the load along the span. Therefore, unless it can be proved that harmonics other than the first may safely be neglected, it will either be necessary to modify the K factors or to calculate the distribution separately for the various harmonics. With the latter process it is possible to obtain a safe answer by assuming that only the first few harmonics are distributed.

The analogy between slabs and beams on elastic supports suggests a simple method of allowing for the effect of a stiff edge beam. If such a beam is considered to be connected to the edge of the slab by a hinge, its only effect will be to exert an upward force on the slab proportional to its stiffness and the deflection of the slab. If the latter would be y_o without the edge beam and y_u is the deflection of the edge due to a unit load (distributed sinusoidally along the span) applied at the edge, the deflection of the edge with the edge beam, y_T, will be given by the difference between the deflection of the unstiffened slab and that due to the reaction of the edge beam on the slab Qy_uy_T, where $Q = EI\pi^3/4a^3$* if only the first term of the Levy series is used, i.e.

$$y_T = y_o - Qy_uy_T = y_o/(1 + Qy_u).$$

This assumes that the beam at the other edge has no effect. If it has, two simultaneous equations, each containing terms for the deflection of both edges, can be derived. Where it is desired to take account of the torsional rigidity of such edge beams the equations will have to contain terms for the rotation of the edges for which formulae are available. The transverse bending moment due to edge beams can be obtained by treating the forces they exert as an additional load on the edges of the slab. As, apparently, stiff edge beams may relieve a slab of a considerable part of the total load to be carried it would be very useful to have experimental confirmation of some method of calculation.

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*The deflected form of a beam of span $2a$ and flexural rigidity EI carrying a distributed load $p.\ sin\ \frac{\pi x}{2a}$ is given by

$$y = \frac{16a^4}{\pi^4 EI}\, p.\ sin\ \frac{\pi x}{2a} \text{ or } \frac{4a^3}{\pi^3 EI}\ P.\ sin\ \frac{\pi x}{2a}$$

if P is the total load. The reaction against a centre deflection y_T is therefore Qy_T.

Mr. L. R. WADDINGTON : Table 5 can lead the unwary into traps (as it led me) if one is dealing with θ-values intermediate between those given. As an example I have plotted on the diagram Fig. 4 in circles, the μ-values given for a section at 0, due to a load at 0, and due to a load at $+ b$ or $- b$, and have drawn reasonable-looking smooth curves through these values, in full lines. These curves, however, are erroneous, as I discovered when I calculated intermediate μ-values from Hetenyi's formulae.[3] The additional points so obtained are shown in triangles, and they indicate the true form of the curves, which are shown in dotted lines. Similarly erroneous curves can be drawn for other section postions and load eccentricities. The possibilities of error would, however, be greatly reduced if a full range of μ-values were available for additional θ-values of about 0.40 and about 1.0.

With regard to Table 6, the form of curves plotted through the given values is such that interpolation for intermediate θ-values should not lead to such gross errors. Nevertheless it is suggested that there is a rather wide gap between $\theta = 0.$ and $\theta = 0.668$ that, to cut down interpolation errors in that important range, additional μ-values should be given for, say, $\theta = 0.30$.

In this table, the μ-value of + 827 at $\theta = 1.057$, and $\frac{b}{4} + \frac{b}{4}$ does not plot on a smooth curve. Would the Authors kindly confirm that there is no error in it? It would also be interesting to know the methods by which these tables were derived.

With regard to the term "r" on p. 95, will the Authors confirm that this is the first harmonic coefficient for a concentrated load, and if so, do they consider it is sufficient to take the first harmonic only for all cases of loading? Will they also state what should be taken for a distributed load?

Finally, I would suggest from experience that the full-torsion curves (Figs. 11 to 15) are difficult to use. Could not the Authors persuade the Cement and Concrete Association to issue them on a larger scale and on much closer mesh graph paper? In fact it would be most helpful to designers if they would issue a uniform series of charts covering all the Figs. 4 to 15 and Tables 5 and 6 as well.

Mr. A. F. MYLES : The Authors are to be congratulated on having presented an interesting and valuable experimental verification of preceding theoretical work.

As the Authors point out, only theoretical grillages and their laboratory equivalents have been studied and therefore at the present stage of the investigations the practising designer must approach the distribution coefficient method in a mood of extreme caution. It might be remarked that this mood can be conveniently engendered by contemplation of the fourth order partial differential equation from which the design curves are derived. I feel that the methods presented in the paper should, at the present moment, be applied only as an informative check to finished designs calculated on the basis of existing experience and methods.

We look forward with interest to the results of further investigations in which the Authors intend to study more practical forms of bridge deck, and these results will be particularly valuable if the curves can be modified to allow for the effect of Poisson's Ratio in slab decks, and the influence of the stiffness of the slab in other forms of construction.

Several times during the presentation of the paper the Authors made the statement that the transverse

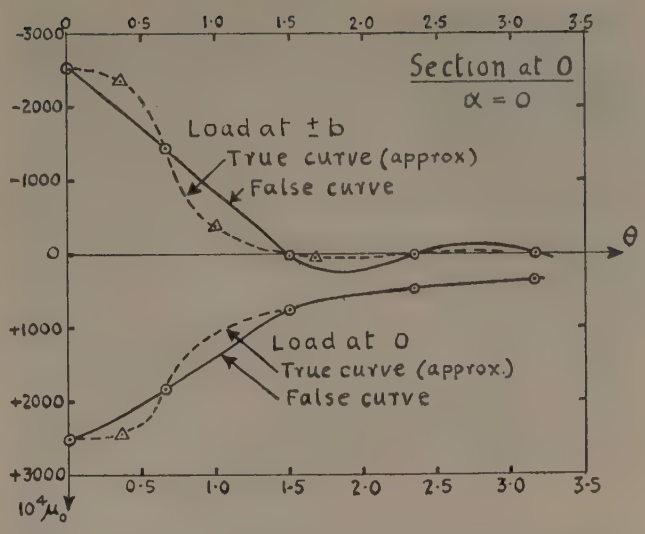

Fig. 4.—Possible errors introduced into transverse bending moment coefficients

bending moment amounted to approximately 50 per cent. of the longitudinal bending moment; this percentage will no doubt strike most designers as surprisingly high. Before any method advocating such a high percentage of transverse strength, with its consequent increase in cost, can be accepted, the reasons for this difference between theory and existing practice must be clearly understood, and the Authors could with advantage discuss this point further. For example, do the Authors in referring to longitudinal bending moment mean the longitudinal bending moment due to the concentrated load system as calculated by the methods of the paper? If so, will not the transverse bending moment expressed as a percentage of the longitudinal design moment be in certain cases much less than 50 per cent.

The methods advocated in the paper are only applicable if it is wished to produce a structure in which concentrated loads will be distributed laterally by means of a resistance to transverse bending. This is an operating condition which the designer will not always wish to produce; in most cases the provision of transverse strength is more costly than an equivalent increase in longitudinal strength and the designer must strike an economic balance between the two. In striking this balance it should be realised that a bridge deck with a much smaller resistance to transverse bending than that required by the methods presented in the paper can nevertheless distribute concentrated loads perfectly satisfactorily by means of torsion and shear. This fact was confirmed by a recent load test on a full scale structure with which the writer has been intimately connected. If the Authors can provide information based on experimental determinations on the subject of load distribution by shear and torsion the value of their future results will be greatly enhanced.

Professor R. G. ROBERTSON (Member) : Owing to some difficulty found in interpreting the results given for the moment in diaphragms in the paper by Dr. Morice and Mr. Little, and since their coefficients for transverse moment increase as the stiffness of the diaphragms increases whereas Dr. Lazarides finds that the weaker the diaphragm the greater is the moment in it[5] and his work is stated to be correct by Dr. Morice, I venture to subscribe an elementary calculation for the maximum possible moment in such diaphragms which

may be of interest in these circumstances. Moreover, Dr. Morice does not give any example showing a calculation of transverse moment, and in his example the diaphragms 14 in. × 5 in. are designed to be stiff enough to restrain the main beam moment, but he gives no calculation for the moment in the diaphragms.

Also there appears to be a mistake on p. 100 in the second column, line 34, where he states that increasing the prestress in a beam increases its stiffness, whereas it can only increase the beam stiffness when the beam is no longer behaving elastically. If a decrease in transverse stiffness is obtained by a decrease in transverse prestress it indicates that the transverse beams are no longer behaving elastically, which would invalidate his calculations.

It is obvious that over-stressing can crush a beam whatever moment it is subject to, and that the stiffness determines the value of I and the maximum range in moment determines the value of the stress, i.e. of $\frac{My}{I}$, and therefore the correct dimensions for the cross beams.

The use of the first harmonic is an approximation which may lead to an excess in the coefficient μ for the transverse moment per unit length due to a central point load P, at any distance along the span $2a$. This is given on p. 95 as $\mu\frac{Pb}{a}\text{Sin}\frac{\pi x}{2a}$, where $\frac{b}{a}$ is the width to span ratio in this case it appears that the total transverse moment is $\frac{2}{\pi}\mu PB$ where B is the total width.

For $\theta = 0$ it is evident that all the beams share the load equally and the total transverse moment must be $\frac{1}{8}PB$, where B is the total width, whereas the Authors appear to give $\frac{1}{2\pi}PB$.

The use of the first harmonic gives 20 per cent. greater moment than the assumption of infinite rigidity for the case of the central point load ; will the Authors state if this is possible for any other cases, for which a note on the maximum possible moments is appended ?

Other cases are given in the attached calculations, for longitudinal beams closely spaced, with diaphragms of infinite transverse rigidity, which must give the maximum possible transverse moment, which is of value for bridges which are narrow in comparison to the span.

Model analysis on small models seems to afford the best method of verifying any more abstruse calculations and the writer would be glad to know if this method had been attempted.

Maximum Moment in a Diaphragm due to Knife Edge Loading

If the diaphragms are infinitely rigid a knife edge linear load W per unit width placed symmetrically on any diaphragm, causes the same reaction in each main beam.

Let B be the total width of the deck. If the load W extends to a distance x each side of centre, the maximum transverse moment in the diaphragm is

$$M = Wx\left(\frac{B}{4} - \frac{x}{2}\right) \quad \ldots\ldots\ldots \quad (1)$$

Maximum moment

occurs when $x = \frac{B}{4}$

$$\text{Maximum } M \quad = \frac{WB^2}{32} \text{ sagging} \quad \ldots \quad (2)$$

When the load covers the whole width of the carriageway, $x = \frac{A}{2}$;

$$M = W\frac{A}{8}(B - A)$$

When the load covers the outer strips of carriageway only of width $\left(\frac{A}{2} - \frac{B}{4}\right)$

$$M = \frac{W}{32}(2A - B^2) \text{ hogging} \quad \ldots\ldots \quad (3)$$

This equals the sagging moment if $A = B$.

Maximum moment in

$$\text{main beams} = WA\frac{L}{4}$$

$$\frac{\text{Maximum Transverse beam moment}}{\text{Maximum total main beam moment}} = \frac{1}{8}\frac{B}{A}\frac{B}{L} \quad (4)$$

Since flexibility in the diaphragm will not alter the total load distributed by the diaphragm, but will increase the relative reactions of the main beams under the loaded portion, the above are the maximum possible moments in any diaphragm, due to load at the position of the diaphragm.*

Uniform Load on Main Beams

Let there be one central diaphragm assumed infinitely rigid, and uniform load w per unit area covering a central strip of the deck of width $2x$.

Let R_1 be the reaction of the load main beams downwards per unit length of diaphragm over the distance $2x$ and let R_2 be the reaction of the outer main beams upwards per unit length of the diaphragm over the two outer distances $\left(\frac{B}{2} - x\right)$

These loads on the diaphragm are in balance.

$$\therefore\ 2R_1x = R_2\,(B - 2x)$$

$$\therefore\ (R_1 - R_2) = \frac{R_2B}{2x}$$

The deflections of all the beams are the same

$$\therefore \frac{5}{384}wL^4 - \frac{1}{48}R_1L^3 = \frac{1}{48}R_2L^3$$

$$\therefore\ (R_1 + R_2) = \frac{5}{8}wL$$

Substituting for $(R_1 + R_2)$ from above :

$$\therefore\ R_2 = \frac{5}{4}\ \frac{wLx}{B}$$

The moment in the diaphragm

$$= \frac{1}{8}\ BR_2(B - 2x)$$

$$= \frac{5}{32}\ wLx(B - 2x) \quad \ldots\ldots \quad (5)$$

Maximum moment in the diaphragm

$$= \frac{5}{32}wLB^2 \text{ when } x = \frac{B}{4} \text{ as before} \quad \ldots \quad (6)$$

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* This is contrary to the results given by T. O. Lazarides. See Civil Engineering and Public Works Review, December, 1952, where the results show an increased tranverse moment due to a decreased tranverse rigidity.

If the carriageway width is A the maximum hogging moment is found when load covers the outer strips and this moment is

$$\frac{5}{256}wL(2A-B)^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

Since the total moment in the main beam is $wA\frac{L^2}{8}$

$$\therefore \frac{\text{Maximum transverse moment in single central diaphragm}}{\text{Maximum total main beam moment}} = \frac{5}{32}\frac{B}{A}\frac{B}{L} \quad . \quad (8)$$

(For the KE load this ratio was $\frac{1}{8}\frac{B}{A}\frac{B}{L}$ by Eq. 4.)

The above moment is reduced when several diaphragms are used.

If diaphragms are closely spaced at distance q the effect under uniform loading is a uniform transfer of load to the outer beams, so that all beams have a uniform loading and the transverse moment in any diaphragm is given from the equation for linear load on each diaphragm Eq. 2 $M=\frac{wqB^2}{32}$ where q is the diaphragm spacing.

If p is the main beams spacing :

$$\frac{\text{Maximum Transverse beam moment}}{\text{Maximum main beam moment}} = \frac{1}{4}\frac{q}{p}\frac{B}{A}\frac{B^2}{L^2} \quad . \quad . \quad (9)$$

These moments are reduced for flexible diaphragms.

Mr. Y. S. Lau : When the bridge is fully loaded by a uniformly distributed load, the distribution coefficient K should be equal to unity everywhere, i.e. there should be no differential vertical deflexion across the bridge, and the transverse B.M. which is proportional to $\frac{\delta^2 w}{\delta y^2}$ must therefore be zero everywhere. In other words, the area of the transverse B.M. influence diagram for any position in the bridge must be zero. Referring to the tables 5 and 6, if we take any horizontal row and apply the trapezoidal rule, i.e. add the sum of all the intermediate values to half the sum of the first and last values in that row, the total so obtained should nearly equal zero, and it should be possible to check the accuracy of the tabular values by this method. For table 5 the totals are (reading downwards) 0, 14, 40, 0, 0 ; 100, 92, 484, 20, 0 ; 226, 217, 173, 81, 0 ; 209, 213, 210, 171, 0 ; 210, 221, 208, 180, 0 ; and for table 6 they are : 0, 0, 0, 0, 0 ; 157, 184, 161, 107, 0 ; 205, 296, 187, 141, 0 ; 203, 248, 196, 168, 0, 199, 198, 195, 171, 0. There is obviously one large spot error in table 5 (row 8, $\theta = 0.669$, load eccentricity $= -b/4$) which has been pointed out by Mr. Goldstein. Quite a number of the totals in the other rows are, however of the order of 200, which appears to be rather large. Would the Authors care to comment ?

The Authors' Replies

Dr. Morice :

Of the many points raised in the discussion there will be seen to be many comments upon transverse moments. In preparing replies we have therefore decided that Mr. Little shall concentrate upon a general note on transverse moments and that I shall cover more general matters.

In the first place there are a number of errors which must be corrected. These are entirely the responsibility of ourselves.

P. 83 Fig. 2—for $\frac{"b"}{2}$ and " b " read $\frac{"p"}{2}$ and " p ".

P. 86 Fig. 6—the curve marked $-\frac{b}{4}$ is incorrect but the curve $-\frac{b}{2}$ of Fig. 5 may be used in its place.

P. 89 Footnote col. 1—for " see page 8 " read " see page 90 ".

P. 91 Figs. 12 and 13—for " Massonnett " read " Massonnet ".

P. 92 Table 1—add to caption $\theta = 1.0$
Table 2—add to caption $\theta = 1.0$
Table 3—add to caption $\theta = 1.0$
Replace figures in col. 6 by —0.08, —0.16, 1.87, 4.85.

Slight differences in the numerical values between Table 2 and Table 3 are due to the values having been read off the curves on different occasions.

Table 4—add to caption $\theta = 0.5$

P. 94 Table 5 $\theta = 0.699$ section $\frac{b}{2}$ load $-\frac{b}{4}$ read " —82 " for " +470 ".
Table 6 $\theta = 1.057$ section $\frac{b}{4}$ load $\frac{b}{4}$ —read " +603 " for " +827 ".

P. 95 Line 23—for " $\mu_\alpha = \mu_0 + (\mu_1 - \mu_0)\sqrt{\mu}$ " read " $\mu_\alpha = \mu_0 + (\mu_1 - \mu_0)\sqrt{\alpha}$ ".

In reply to Mr. Chettoe's contribution I should like to thank him for adding weight to the necessity for using this kind of analysis in practice when the relatively new concentrated abnormal loads have to be catered for. We have advocated the distribution coefficient method because we feel that it is the most satisfactory for design office use whilst giving excellent agreement with experiment. The method is based upon elasticity theory and, as Mr. Chettoe says, may be applied to any structural material or combination of materials which is elastic.

Mr. Chettoe has mentioned a most important point when he states that the engineer must ensure that the structure behaves as assumed. The greatest strength of a bridge deck will be obtained when the longitudinal beams become non-linear first since the effect will be to increase load distribution. Primary failure of the diaphragms will obviously greatly reduce the distribution, and thus the intrinsic ultimate strength.

I am indebted to Mr. Goldstein for his reference to Engesser's work. Whilst this approximation may give a not unreasonable assessment of the longitudinal moments I think it would be well out for diaphragm stresses. In attributing work to various authors I feel that mention must be made of the early work of Guyon[2] on slabs.

I cannot but agree with Mr. Goldstein that the optimum value of θ will be that giving the cheapest bridge, which may well vary from site to site and with the form of the maximum design loading. Unfortunately we have little opportunity to obtain such

information and must leave this to Consultants. However, as Mr. Henderson has pointed out, the relative ease with which the effects of changes in proportions may be noted should make the preparation of alternative schemes a possibility.

The difficulty of obtaining a theoretical solution to the case of a skew bridge lies in the fact that the form of the deflexion function $w(x, y)$ is not separable into the form $\psi(x).\psi(y)$ which results in very great simplifications. Application of energy methods, with approximations, may yield a solution ; this is being investigated. The problem has similar analytical complications to that of the instability of a plate under shear.[6] Relaxation methods may be applied but by this means each problem requires a separate fairly lengthy computation.

In reply to two further points raised by Mr. Goldstein I would say that we have used only the first term of the harmonic series and that a new set of distribution coefficients would be required for each term. The case of a vierendeel diaphragm would seem to need experimental investigation, but I should be inclined to use the smaller stiffness, until further information is forthcoming.

Mr. Harris has noted the change with prestress in stiffness of some of our diaphragms. This we attribute to the fact that no cracks occurred with the higher prestress. The question as to the direction of the diaphragms in a skew bridge is difficult since we have not, as yet, carried out any experimental work and, as I noted above, no analytical theory exists. From the practical point of view I feel that the similarity of all longitudinal beams when skew diaphragms are used would be helpful, also, although of doubtful applicability here, it may be noted that aeronautical engineers prefer oblique wing ribs in swept wings, this being a somewhat similar problem. In the case of our simply supported test specimens the inclusion of support diaphragms would undoubtedly have increased the torsional stiffness and would be good practice.

Concerning Mr. Henderson's remarks I should like to emphasize again that the greatest ultimate strength will be obtained if the distribution can be increased rather than reduced at failure. The diaphragms must therefore be strong enough to force non-linearity in the main beams even though their worst condition of loading does not cause the greatest moment in the longitudinal beams. A case may exist here for a more sophisticated design where the inner longitudinals are less strong than the edge beams. The analysis of such structures may well be tackled upon the lines indicated by Mr. Spindel.

The proportions of transverse moments to longitudinal moments to which Mr. Holland draws attention were for the unpractical case of a point load. In the case of a reasonably distributed load, such as the M.O.T. abnormal train, the value would be of the order of 43 per cent. although this will depend upon the slab proportions. For general traffic loading both the longitudinal and transverse moments will be well below the design values and I would suggest that this is why Mr. Holland has had no trouble in the past. Indeed it is only because these large, relatively concentrated, loads are now in existence that the problem has become really important.

I fear that I must disagree with Mr. Myles on several points. In the first place I feel that the method, if used at all, has its greatest value in the early stages of design where the effects of proportioning of the structure are required to be known. Secondly I believe that wrong thinking is leading Mr. Myles astray. The amount of distribution in an elastic structure will depend upon the amount of transverse stiffness which is built in. If Mr. Myles finds it cheaper to carry load with little distribution then appropriate proportioning of his structure will do this. In the extreme we shall have the case of Mr. Goldstein's Fig. 1a. However, if greater transverse stiffness exists then either there will be developed the larger transverse moments or if the transverse strength is insufficient cracks will develop and the effective transverse stiffness will be reduced. What must be made clear is that if adequate transverse strength is not available then the distribution will be less than that calculated from the theoretical transverse stiffness, and the structure will not be behaving elastically.

Finally, it must be pointed out that in the case of slabs the torsional stiffness factor to be substituted into the expression for α is that coming from the theory of plates, viz. : $\frac{d^3}{6}$

Mr. Little :

Many questions have been asked about transverse bending moments. The following procedure for the determination of My is due mainly to Guyon. We now feel that the method of analysis based on the ratio of transverse to longitudinal bending moments has not yet been investigated with sufficient thoroughness to form a general method of design although the value of 50 per cent. stated by Mr. Rowe has been confirmed for a single concentrated load acting on a slab bridge having a value of the parameter $\theta = 0.4$.

The total transverse bending moment My is found by taking the sum of the partial moments

$$b\,\mu_n\, r_n \operatorname{Sin}\frac{n\pi x}{2a} \text{ where } n = 1, 2, \ldots \infty$$

Thus :

$$My = b\left[\mu_{(1)}r_1 \sin\frac{\pi x}{2_a} + \ldots \mu_{(n)}r_n \sin\frac{n\pi x}{2_a} + \ldots\right]$$

Values of μ_o (i.e. μ for $\alpha=0$) as a function of θ were given in Table 5 of the paper whilst values of μ_1, (i.e. μ for $\alpha=1$) appeared in Table 6.

If the series is limited to the first term then

$$My = b\,\mu_{(1)}r_1 \sin\frac{\pi x}{2a}$$

where $\mu_{(1)}$, corresponds to the bridge under analysis having a stiffness ratio defined by θ, and $r_1 \sin\frac{\pi x}{2a}$ is the first term of the load series.

If we cannot limit ourselves to the first term of the series it is to be noted that the coefficient $\mu_{(n)}$ is simply the value of μ given by the table corresponding to a bridge having a value of θ, n times greater than that for the actual bridge. In other words the total moment is the sum of a series whose successive terms are the partial moments produced by the successive terms of the load function on bridges once, twice, . . . n times more flexible than the actual bridge under analysis.

Thus, $\mu_{(1)}$, $\mu_{(2)}$, . . . $\mu_{(n)}$ can be written μ_θ, $\mu_{2\theta}$, ... $\mu_{n\theta}$ . . . and the total moment is given by

$$My = b\left[\mu_\theta r_2 \sin\frac{\pi x}{2a} + \ldots\ldots \mu_{n\theta}r_n \sin\frac{n\pi x}{2a} + \ldots.\right]$$

This is valid for values at both $\alpha = 0$ and $\alpha = 1$.

Almost invariably the maximum transverse moment will occur at the point on the bridge defined by $x = a$ and $y = 0$.

Thus in Tables 5 and 6 the most useful values of μ for any value of θ are those for the position of section 0. To help in constructing influence curves for this section further values of $\mu_0 \cdot 10^4$ and $\mu_1 \cdot 10^4$ are given in Table 1 for intermediate values of θ. If the values of these rows are plotted across the transverse section at any value of θ the resulting curve is the influence line for μ for the central section. To get the effect of a number of wheel loads acting over a transverse section it is sufficient to sum the ordinates of the influence curve for μ at the points of action of the loads. The value of P to be used in the calculation of the load series is then the load on one wheel.

To assist in the calculation of the bending moments Table 2 has been constructed for various longitudinal dispositions of load. The function $\rho_n = r_n \sin \frac{n\pi x}{2a}$ corresponds to the partial load term given above.

Thus :

$$My = b \, \Sigma \mu_{(n)} r_n \sin \frac{n\pi x}{2a}$$

It is evident that the number of terms to be taken depends on a number of parameters. It can be seen that the convergence of the load series varies with the disposition of the load being least for a concentrated load since the factor $\frac{1}{n}$ does not occur in the expression for r_n. Normally we are mostly interested in the values of My at the centre of the longitudinal span, i.e. $x = a$.

Then :

$$\rho_n = r_n \sin \frac{n\pi}{2}$$

Thus, for a concentrated load of longitudinal position u ;

$$P_n = \frac{P}{a} \sin \frac{n\pi u}{2a} \sin \frac{n\pi}{2}$$

For a load at mid span

$$\rho_n = \frac{P}{a} \sin^2 \frac{n\pi}{2}$$

Here it is noted that $\sin^2 \frac{n\pi}{2}$ is equal to 1 if n is odd and o if n is even.

The value of My is thus :

$$My = b \frac{P}{a} \left[\mu_\theta + \mu_{3\theta} + \mu_{5\theta} + \ldots \right]$$

The convergence of this series will depend upon the value of θ since the greater is θ the sooner will the value of $\mu_{n\theta}$ reach small values which can be neglected.

For example, for equal loads at sections $\pm \frac{b}{4}$ and $\pm \frac{b}{2}$ with $\theta = 0.699$ and $\alpha = 0$; (Note : that if the influence line for μ at section 0 is constructed it is not necessary to distribute the loads to the standard positions.)

$$\mu_\theta = 0.669 = 2\left[0.075 - 0.010 \right] = 0.130$$

$$\mu_{3\theta} = 2.007 = 2\left[-0.006 - 0.006 \right] = -0.024$$

which is sufficient to define My.

$$My = b \frac{P}{a} \left[0.130 + 0.024 \right] = 0.106 \frac{Pb}{a}$$

where P is the load on one wheel.

In the case where $\alpha = 1$ i.e. for a slab

$$\mu_\theta = 0.669 = 2\left[0.0213 - 0.0151 \right] = +0.0124$$

$$\mu_{3\theta} = 2.007 = 2\left[-0.005 - 0.004 \right] = -0.0180$$

$$\mu_{5\theta} = 0 \quad \text{(very nearly)}$$

$$My = b \frac{P}{a} \left[+0.0124 - 0.0180 \right] = -0.0056 \frac{Pb}{a}$$

It is to be noted how the transverse disposition of the load has a great effect on the number of terms that must be taken when the sign of the value of μ is different for the different individual loads, i.e. $b/4$ and $b/2$ in this case.

Poisson's Ratio Effect

The values of μ so far discussed have been calculated on the assumption that Poisson's ratio, $\sigma = 0$. Whilst this assumption is always permissible in actual structures when $\alpha = 0$ we must take into account the effect of Poisson's ratio in all cases for which $\alpha \neq 0$.

Rather than work out a complete set of μ_3 values to make this correction we have found by experiment that when $\alpha = 1$ i.e. for a slab, it is sufficient to increase the tabulated μ_1 values by approximately 15 per cent.

It is felt that this correction will be quite adequate in practical design in spite of not being strictly accurate.

The effect of Poisson's ratio will decrease as α decreases and will be represented at any given value of α by

$$\Delta \mu_\alpha = 15\sqrt{\alpha} \text{ per cent.}$$

This is the amount by which the calculated values of μ_α must be increased to give the real value of My.

Thus,

$$My \ (theoretical) = My_0 + (My_1 - My_0)\sqrt{\alpha}$$

$$My \ (actual) = My \ (theoretical)(1 + 0.15\sqrt{\alpha})$$

My with longitudinal disposition of loads

If more than one transverse section is loaded, as would be the case when the M.O.T. abnormal load acted on a bridge, the value of My is found by superposing the My moments caused by each row of loads.

This can be done by using equation (5) of Table 2.

Then, for equal wheel loads P

$$My = b \Sigma \mu_n \frac{P}{a} \left(\sin \frac{n\pi u}{2a} + \sin \frac{n\pi u''}{2a} + \ldots \right) \sin \frac{n\pi x}{2a}$$

When u' and u'' are the distances of the transverse lines of load from the end of the bridge. If we consider

the transverse moment My at mid span the function $sin \frac{n\pi x}{2a}$ will become $sin \frac{n\pi}{2}$ in the expansion.

Questions have been asked regarding the high values of the ratio My/Mx which we have obtained for a slab. The value of 50 per cent. occurred in a slab for which $\theta = 0.4$. A recent tentative theoretical determination of the ratio has indicated that the ratio would be expected to increase with θ being governed by the relation

$$\frac{My}{Mx} = f\left(\frac{\mu}{k}\theta^2\right)$$

Current tests for $\theta = 0.5$ have shown an increase in the $\frac{My}{Mx}$ ratio from 50 per cent. (at $\theta = 0.4$) to 54 per cent. (at $\theta = 0.5$).

It is, of course, possible in a practical structure to have a ratio of moments which is very much smaller. This will usually occur when α is small and θ high. It is not yet possible to formulate a general expression for ratio $\frac{My}{Mx}$ which will govern any type of bridge and it will be better to use the method given above to calculate My directly. Although it is obvious that this method is not particularly suitable for very low values of θ since the convergence of $\mu_{(n)}$ is then very slow, practical structures are included in the higher range of θ . In particular, the non-convergent case of $\theta = 0$ for which $\mu_{(1)} = \mu_{(2)} = \ldots \mu_{(n)} \ldots$ need not be considered.

Then :

$$My = b\mu_{(1)}\, P(\alpha)$$

In the particular case of a concentrated load acting on a very stiff bridge, the total moment is given by

$$b\mu_{(1)}P = \frac{1}{8}(2b)P$$

Which agrees with Professor Robertson's calculations.

Whilst I am in general agreement with his analysis for infinite cases its use cannot be advocated to designers. As it is certain that the moment My decreases with increase in θ the values obtained by his method would often be much greater than required in the design of a bridge.

Further, whilst the maximum value of μ for $\theta = 0$ is obtained for a uniformly distributed load acting between the limits $\frac{-b}{2}$ and $\frac{+b}{2}$ it is to be remembered that these limits decrease appreciably as θ increases to normal values and that the calculations will usually be required for the concentrated wheel loads of the M.O.T. abnormal load. This being so, the method outlined above seems to possess the greatest advantages.

Professor Robertson is, of course, correct in his assumption that an increase in the transverse prestress could affect the effective transverse rigidity only when the structure was behaving non-elastically in the transverse direction. This point has always been considered only for ultimate conditions when the increased transverse prestress leads to the undesirable behaviour noted in the paper.

Small plastic model analysis has been employed to investigate the behaviour of a box-section bridge and a model of a T-deck is ready for test.

A present study of the theory of dimensional analysis will, we hope, extend the scope of the model tests still further.

Most of the points raised by Mr. Myles have been answered by Dr. Morice but I should like to discuss the behaviour of the bridge to which I think he is referring.

It is not denied that as yet there are structures for which the parameters θ and α cannot be defined exactly not because of faults in the method but because of ambiguity in the form of the bridge.

I believe that the bridge to which he refers is one with which I was intimately connected and which fell into this class of structure. It is a bridge which is formed by stressing together a number of precast beams transversely. The transverse moment of resistance is then defined by the maximum compressive stress induced by the prestress. For this bridge the transverse resistance was very small compared with the longitudinal resistance. The test was made with the M.O.T. abnormal load trailer of 110 tons. The measured deflected profiles revealed that for most loading positions the induced My moments had exceeded the resistance moment at the section of measurement.

The distribution deteriorated as the load approached this section and was always decidedly inferior to the calculated distribution. However, as this was a continuous structure, there was one position of load such that the transverse resistance would either not be exceeded or only slightly exceeded. The distribution then was better than the Guyon distribution yet inferior to the slab distribution. A value of α has been found which defines the actual behaviour. Since the structure is continuous transversely when the transverse moment of resistance is not exceeded the theoretical value of α is dependent upon the normal pressure between adjacent beams and, hence, to the transverse prestress. The effective value of α will increase as the pressure increases always approaching the condition of a slab i.e. the degree of twist between beams will approach that in a continuous slab. If the calculated prestress were used the bridge should be described by the Massonnet analysis. It is to be noted that as long as the induced transverse moment is less than the transverse moment of resistance then the value of θ will remain unchanged irrespective of the value of α .

Most of Mr. Waddington's points have already been considered in the previous discussion. The value of $\mu \cdot 10^4$ for $\alpha = 1$ and co-ordinates $\frac{b}{4}, \frac{b}{4}$ has been recalculated. The true value is + 603 rather than + 827.

We are at present checking the values of μ included in Tables 5 and 6 to obviate the discrepancies noted by Mr. Lau. This is rather a lengthy business but the results will be made generally available immediately they are obtained. However, we have found that the discrepancies are not of the same order as given by Mr. Lau. This is because of the extreme care which must be taken when using the mid-ordinate rule for profiles which have a rapid rate of increase of ordinate with length. This is especially apparent in the region of $-\frac{b}{4}, 0, +\frac{b}{4}$. By taking the actual area of the μ influence curves the errors are found to be much

smaller than those stated and are of the order of 20 rather than 200.

I am grateful to Mr. Spindel for his interesting contribution. Of special interest is his reference to the Hetenyi solutions for bars (or beams) with lateral and axial loading. The two " finite " solutions given by Hetenyi for equal loads and equal bending moments at each end of the bar should be useful to our future investigations into the influence of non-axial prestressing in the transverse direction on the initial state of a bridge, and also, as he says, in assessing the influence of stiffened edge beams.

The assumption that the same K factors be used for bending moments as for deflexions i.e. using only the first harmonic in their derivation, can certainly be made for low values of θ. The accuracy of the assumption decreases with an increase in the value of θ. However, rather than extend the method to the use of further harmonics it has been found experimentally, and stated in theory (Guyon) that in the normal range of θ, say 0.4 to 0.7, it is sufficient to increase the theoretical bending moment based on the first harmonic by some 15 per cent. The accuracy will then be of the same order as for deflexion.

Table 1

Values of $\mu_0 \cdot 10^4$ (i.e. for $\alpha = 0$)

Section	θ	Load Position							
		$-b$	$-\frac{3b}{4}$	$-\frac{b}{2}$	$-\frac{b}{4}$	0	$\frac{b}{4}$	$\frac{b}{2}$	$\frac{3b}{4}$	b
0	0.2	—2475	—1240	— 15	+1225	+2540	+1225	— 15	—1240	2475
	0.4	—2280	—1160	— 30	+1150	+2370	+1150	— 30	—1160	— 2280
	0.8	— 930	— 560	—160	+ 520	+1520	+ 520	—160	— 560	— 930
	1.0	— 390	— 330	—180	+ 260	+1160	+ 260	-180	330	390
	1.2	— 160	— 200	—180	+ 80	+ 920	+ 80	—180	— 200	— 160

Values of $\mu_1 10^4$ (i.e. for $\alpha = 1$)

Section	θ	Load Position							
		$-b$	$-\frac{3b}{4}$	$-\frac{b}{2}$	$-\frac{b}{4}$	0	$\frac{b}{4}$	$\frac{b}{2}$	$\frac{3b}{4}$	b
0	0.2	—1760	—960	— 62	+910	+2080	+910	— 62	—960	—1760
	0.4	—1020	—600	—130	+550	+1580	+550	—130	—600	— 1020
	0.8	— 280	—240	—140	+140	+ 940	+140	-140	—240	— 280
	1.25	— 60	— 80	— 90	0	+ 610	0	— 90	— 80	— 60

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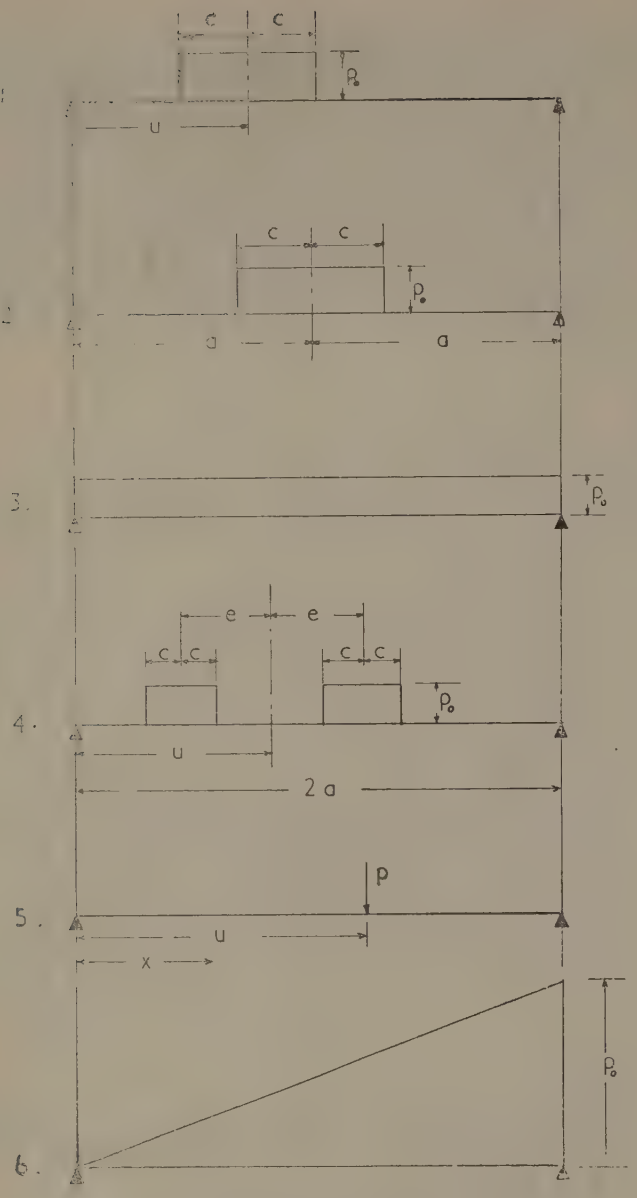

Fig. 5.—Load series to be used for transverse moment determination

Table 2.

Load series to be used for transverse moment determination" (see Fig. 5)

$$\rho_n = r_n \sin \frac{n\pi x}{2a}$$

1. $$r_n = \frac{4\rho_o}{\pi n} \sin \frac{n\pi u}{2a} \sin \frac{n\pi c}{2a}$$

2. $$r_n = \frac{4\rho_o}{n\pi} \sin \frac{n\pi}{2} \sin \frac{n\pi c}{2a}$$

3. $$r_n = \frac{4\rho_o}{n\pi} \sin^2 \frac{n\pi}{2}$$

4. $$r_n = \frac{8\rho_o}{n\pi} \sin \frac{n\pi u}{2a} \sin \frac{n\pi c}{2a} \cos \frac{n\pi e}{2a}$$

5. $$r_n = \frac{P}{a} \sin \frac{n\pi u}{2a}$$

6. $$r_n = -\frac{2\rho_o}{n\pi} \cos n\pi$$

Book Reviews

(continued)

Practical Design of Simple Steel Structures, Vol. 2, 3rd Edition, by D. S. Stewart. (London : Constable, 1953.) 297 plus xiii pp., 245 figs., xi plates 8½ in. × 5½ in. Price 25s.

This volume, which deals with girders, columns, trusses, bridges, etc., has been revised to conform with the latest B.S.S. The book is set out in the same way as the previous edition, the calculations and explanatory text of each of the complete designs being kept apart on opposite pages. The worked examples include the design of a 40 ft. span gantry girder, a 55 ft. span roof truss, a lattice girder footbridge, a 70 ft. span through railway bridge and a 70 ft. span road bridge. Welded design and construction have not been included as this could not be dealt with adequately in the space available.

The book has been a useful guide for students and engineers for a number of years, and the new up-to-date edition will be welcomed.

Theoretical Naval Architecture, by E. L. Attwood and H. S. Pengelly, revised by A. J. Sims. (London : Longmans, Green, 1953.) 8½ in. × 5½ in., 418 plus xii pp., 1 plate and 2 folding tables. Price 40s.

This volume is an expansion of the " Text-Book of Theoretical Naval Architecture," originally published in 1899. The 1931 edition has been completely revised, many chapters rewritten and new chapters added to bring the book up-to-date. As in previous editions, many examples are given in the text and at the end of each chapter, and standard tables are included in the Appendices.

Institution Notices and Proceedings

IMPORTANT NOTICE TO SUBSCRIBERS

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ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 25th November, 1954, at 5.55 p.m., Dr. S. B. Hamilton, M.Sc., Ph.D., B.Sc.(Eng.), A.R.C.S., M.I.C.E., M.I.Struct.E. (President) in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

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EVANS, Herbert Gordon, of Bexley, Kent.

OBITUARY

The Council regret to announce the deaths of Samuel Grange DUCKWORTH, William ELLIS-JONES, Charles Edmond FONSEKA, George Willis GREY, Alexander Burns LAWSON, Edwin NEWTON, Major Vivian Edward PURCELL, Frederick Robert RUDKIN (Members); Hedley Cecil QUEREE (Retired Member); Professor John ORR (Honorary Associate); Ralph Elston MILES, M.C. (Associate); Justin James BEHRENS, Frederick Charles BRIDLE, George William DICKIN, Alfred Eaton GOODWIN, John POLLARD, Frank James SMITH, Harry WEST, George WHITESIDE, Charles Leonard COMMON (Associate-Members); Leslie CREIGHTON, Ronald HEDGECOCK (Graduates); and Brian PERKIN (Student).

EXAMINATION PRIZE LIST, JULY, 1954

The Council have awarded the following prizes in connection with the examinations held in July, 1954 :—

ANDREWS PRIZE (For the candidate who obtains the highest aggregate of marks in the Associate-Membership Examination, passing in all subjects)

Frederick Ernest WOTTON, of London.

HUSBAND PRIZE (For the candidate who takes the whole of the Associate-Membership Examination, passes in all subjects, and obtains the highest marks in the paper "Structural Engineering Design and Drawing")

Robert Ross BROWN, of Harrogate.

WALLACE PREMIUM (SENIOR) (For the candidate who takes the whole of the Associate-Membership Examination, passes in all subjects, and obtains the highest marks in the paper "Theory of Structures (Advanced)")

Shriniwas Rangnath KULKARNI, of India.

WALLACE PREMIUM (JUNIOR) (For the most successful candidate in the Graduateship Examination, passing in all subjects)

Eugene Marc Alexander BAIKOFF, of Bergvlei, Transvaal, South Africa.

EXAMINATIONS—JANUARY, 1955

The Examinations of the Institution will next be held at centres in the United Kingdom and Overseas on January 11th and 12th, 1955 (Graduateship), and January 13th and 14th (Associate-Membership).

JOURNAL CASES AND BINDING, 1954

A binding case can be supplied for the twelve issues of the Journal, January-December, 1954 (Volume 32), price 11s. 6d., post free. The price for binding in half-leather, inclusive of packing and postage, is 27s. per volume.

It is requested that all parcels containing Journals forwarded for binding should bear the name and address of the sender and must be despatched to reach the Institution by the 4th April, 1955.

An Index will be included in all bound volumes. Members making their own arrangements for binding may obtain a copy of the Index upon application to the Secretary.

Copies of the Index will be provided for subscribers and those on the complimentary list.

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, January 13th, 1955

Ordinary Meeting, 6 p.m., when Mr. W. R. Garrett, A.M.I.C.E., A.M.I.Struct.E. (Associate-Member of Council) will give a paper on "Gasholder Development and Design."

Thursday, January 27th, 1955

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. G. M. Boyd, A.M.I.Struct.E., M.I.N.A., will give a paper on "Brittle Fracture Problems in Steel Construction."

Thursday, February 10th, 1955

Ordinary Meeting, 6 p.m., when Mr. F. J. Samuely, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., will give a paper on "Structural Prestressing."

Thursday, February 24th, 1955

Ordinary General Meeting, 5.55 p.m., followed by a Joint Meeting with the Cement and Concrete Association and the Reinforced Concrete Association at 6 p.m., when Professor Pier Luigi Nervi will give a paper on "Some Reinforced Concrete Structures in Italy." Owing to the large attendance expected, this meeting will be held at the Central Hall, Westminster.

Thursday, March 24th, 1955

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Dr. E. H. Bateman, M.A., B.Sc., M.I.C.E., M.I.Struct.E., A.M.I.Mech.E. (Member of Council) will give a paper on "The Development of the Elastic Theory of Continuous Frames."

Members wishing to bring guests to the Ordinary and Joint Meetings referred to above are requested to apply to the Secretary for tickets of admission.

DRURY MEDAL AWARD

The fifth competition for the above award will take place in 1955. The subject is the design of a mobile crane.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary: envelopes to be marked in the top left-hand corner, "Drury Medal Award."

The closing date for the competition is October 1st, 1955.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say, primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution, or failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate in the presence of a Justice of the Peace or a Commissioner for Oaths.

INVITATION TO SUSSEX MEMBERS

The paper on " The Design and Construction of the Nuclear Reactor Buildings at Windscale Works, Sellafield " which Mr. D. R. R. Dick gave before the Institution on November 25th is to be repeated at a meeting of the Southern Association of The Institution of Civil Engineers to be held at Brighton Technical College on Thursday, March 24th, 1955, at 6 p.m. Members of the Institution of Structural Engineers resident in Sussex who may have been unable to hear the paper in London are cordially invited to attend this meeting and will be welcome to join members of the Association for tea at 5.30 p.m. In order that catering arrangements may be made, members who intend to be present are requested to notify the Honorary Secretary, Mr. I. Wilson, B.Sc., A.M.I.C.E., City Engineer & Surveyor's Office, " Greyfriars," Chichester, Sussex.

LONDON GRADUATES' AND STUDENTS' SECTION

The following meetings have been arranged :—

Wednesday, January 19th, 1955

The Annual Dance will be held at The Paviour's Arms, Page Street, London, W.1 on Wednesday, January 19th, at 7.30—11.15 p.m. Tickets, 7s. 6d. single and 10s. 6d. double, may be obtained from the Honorary Secretary.

Tuesday, February 15th, 1955

Address by the President of the Institution, 11, Upper Belgrave Street, London, S.W.1., 6 p.m.

Tuesday, March 15th, 1955

Annual General Meeting, followed by a film display, 11, Upper Belgrave Street, London, S.W.1., 6 p.m.

Hon. Secretary : J. A. Pope, 53, Cranleigh Drive, Leigh-on-Sea, Essex.

ADDITIONS TO THE LIBRARY

The following volumes have been presented by Mr. J. A. Cole, M.I.Struct.E. :—

Barets, J.	Le Béton Précontraint—Theorie et Calculs. Paris : 1950.
Proceedings of	International Congress on Prestressed Concrete. Ghent : 1951 ; London, 1952.
Komendant, A. E.	Prestressed Concrete Structures. New York & London : 1952.
Magnel, G.	Prestressed Concrete (3rd Ed.). London : 1954.
Walley, F.	Prestressed Concrete Design and Construction. London : 1953.
Zerna, W.	Spannbeton : Eine Einfuhrung in seine Theorie (Prestressed Concrete : An Introduction to the Theory). Dusseldorf : 1953.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Friday, January 7th, 1955

At the College of Technology, Manchester, 6.30 p.m. Dr. P. W. Rowe, B.Sc., A.M.I.C.E. (Graduate), on " The Present Situation on Retaining Wall Design."

Tuesday, January 11th, 1955

A Joint Meeting with the Institute of Welding (Liverpool and District Branch) will be held at the Liverpool City College of Technology, Liverpool, at 7 p.m., when Mr. L. Rotherham will give a paper on " Welding at the Atomic Plants."

Tuesday, January 25th, 1955

At the College of Technology, Manchester, 6.30 p.m. Dr. T. Howarth, A.R.I.B.A., on " Modern Architecture."

Tuesday, February 1st, 1955

Joint Meeting with the Reinforced Concrete Association, at the Liverpool Engineering Society, Dale Street, Liverpool, at 6.30 p.m. Dr. D. D. Matthews, M.A., D.Eng., M.Sc.(Eng.), M.I.Struct.E., A.M.Am.Soc.C.E. (Member of Council), will give a paper on " Prestressed Concrete Bridge at Nottingham."

Wednesday, February 2nd, 1955

The above meeting will be repeated, commencing at 6.45 p.m.

Monday, February 14th, 1955

Joint Meeting with the Institution of Civil Engineers, at the College of Technology, Manchester, at 6.30 p.m., when Mr. T. C. Waters, M.I.Struct.E., will give a paper on " The Process Building at Capenhurst."

Monday, February 28th, 1955

At the College of Technology, Manchester, at 6.30 p.m. Mr. D. R. R. Dick, B.Sc., M.I.C.E., on " The Design and Construction of the Nuclear Reactor Buildings at Windscale Works, Sellafield."

Wednesday, March 30th, 1955

Joint Meeting with the Institute of Welding (Manchester Branch), College of Technology, Manchester, at 6.30 p.m. Mr. E. Lightfoot, M.Sc.(Eng.), A.M.I.C.E., A.M.I.Struct.E., will give a paper on " Dynamic Stresses in Structures."

Joint Hon. Secretaries : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire ; M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford, 6, Lancs.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged:—

Tuesday, January 4th, 1955

Joint Meeting with the Reinforced Concrete Association, Midland Counties Branch, at the Midland Institute, Birmingham, 6 p.m. Dr. F. G. Thomas, Ph.D., B.Sc., M.I.C.E., M.I.Struct.E. (Member of Council), on " Load Factor Methods of Design of Reinforced Concrete."

Friday, January 28th, 1955

Joint Meeting with the Institute of Welding, Midland Section, at the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m. Mr. S. M. Reisser, B.Sc., M.I.Struct.E., A.M.I.C.E., will give a paper on " The Influence of Welding on Steel Building Structures with Particular Reference to Erection."

Tuesday, February 8th, 1955

At the Supper Room, The King's Hall, Queen Street, Derby, at 7 p.m. Mr. F. Brooksbank, M.A.(Cantab.) (Graduate), on " Economics in Welding Design."

Friday, February 25th, 1955

At the Department of Civil Engineering, The University, Birmingham, at 6 p.m. Professor S. C. Redshaw, Ph.D., D.Sc., M.I.C.E., F.R.Ae.S., on " Modern Methods of Experimental Stress Analysis in Structural Engineering."

Wednesday, March 9th, 1955

At Loughborough College of Technology, Loughborough, at 7 p.m. Dr. J. M. Harvey, Ph.D., M.Sc., and Mr. E. Griffin, on " The Design and Manufacture of Cold Formed Steel Sections as Structural Members."

Friday, March 25th, 1955

At the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m. Mr. V. H. Lawton, M.I.Struct.E., on "Some Notes on the Intended Revisions to B.S.449."

Hon. Secretary: L. A. Firminger, A.M.I.Struct.E., 356, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES GRADUATES' AND STUDENTS' SECTION

The following meetings have been arranged :—

Monday, January 31st, 1955

Joint Meeting with the Junior Section of the Birmingham and Five Counties Architectural Association, at the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m. Mr. H. V. Hill, M.Sc., A.M.I.C.E., A.M.I.Struct.E. (Associate-Member of Council), will give a paper on "The Use of Light Alloys in Structures."

Wednesday, March 30th, 1955

At the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m. Mr. C. B. Brewington, B.Sc., A.M.I.C.E. (Graduate), on "Load Bearing Walls."

To be followed by Annual General Meeting.

Hon. Secretary: A. K. A. Costain, A.M.I.C.E., 134, Witherford Way, Weoley Hill, Birmingham, 29.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Wednesday, January 12th, 1955

Joint Meeting with the Institution of Civil Engineers, at the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, at 6.30 p.m., when Dr. A. W. Skempton, D.Sc., A.M.I.C.E., will give a paper on "Some Practical Applications of Soil Mechanics."

Thursday, January 13th, 1955

Joint Meeting with the Northern Architectural Association, at Higham Place, Newcastle, at 7.30 p.m.

Tuesday, February 1st, 1955

At the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, at 6.30 p.m. Mr. F. A. Partridge, B.Sc., A.C.G.I., M.I.C.E., on "The Plastic Theory."

Wednesday, February 2nd, 1955

The above meeting will be repeated at the Neville Hall, Westgate Road, Newcastle upon Tyne, at 6.30 p.m.

Tuesday, March 1st, 1955

At the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, at 6.30 p.m. Mr. M. F. Palmer, M.I.C.E., M.I.Struct.E., on "Fabrication and Erection of Steel Bridges."

Wednesday, March 2nd, 1955

The above meeting will be repeated at the Neville Hall, Westgate Road, Newcastle upon Tyne, at 6.30 p.m.

Tuesday, March 16th, 1955

At the Neville Hall, Westgate Road, Newcastle upon Tyne, at 6.30 p.m.

Ladies' Guest Night.

Wednesday, March 17th, 1955

At the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, at 6.30 p.m.

Ladies' Guest Night.

Hon. Secretary: Captain O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, January 4th, 1955

At the College of Technology, Belfast, at 6.45 p.m. Mr. J. D. Boyd, B.Sc., will give a paper on "Reinforced Concrete Portal Frames in Northern Ireland."

Tuesday, February 1st, 1955

At the College of Technology, Belfast, at 6.45 p.m. Mr. D. V. Pike, M.I.Struct.E., A.M.I.C.E., will give a paper on "Structural Uses of Aluminium."

Tuesday, March 1st, 1955

At the College of Technology, Belfast, at 6.45 p.m. Film Evening.

Hon. Secretary: A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., "Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Tuesday, January 18th, 1955

At the Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, at 7 p.m. Mr. Hugh Fraser, B.Sc., M.I.Struct.E., A.M.I.C.E., and Mr. W. G. Cantlay, B.Sc., A.M.I.C.E., A.M.I.Struct.E., will give a paper on "A Graphical Approach to the Design of Two and Three Span Rigid Frame Buildings."

Friday, February 11th, 1955

Joint Meeting with the Glasgow and West of Scotland Association of The Institution of Civil Engineers, at The Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, at 7.15 p.m., when Mr. F. A. Partridge, B.Sc.(Eng.), A.C.G.I., A.M.I.C.E., will give a paper on "The Plastic Theory as Applied to the Design of Mild Steel Structures."

Tuesday, March 29th, 1955

At the Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, at 7 p.m. Mr. D. R. R. Dick, B.Sc., M.I.C.E., on "Design and Construction of Nuclear Reactor Building, Windscale Works, Sellafield."

Hon. Secretary: G. Drysdale, A.M.I.Struct.E., "Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, February 18th, 1955

At Exeter, at 7 p.m. Mr. F. E. Somerset, on "The Efficiency of the Tube Section." (Accompanied by a film.)

Friday, March 18th, 1955

At Plymouth, at 7 p.m. Mr. D. R. R. Dick, B.Sc., M.I.C.E., on "Design and Construction of the Nuclear Reactor Building at Windscale, Cumberland." (Mr. S. G. Silhan, A.M.I.Struct.E., will read the paper on behalf of the Author.)

Joint Hon. Secretaries: E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay; C. J. Woodrow, "Elstow," Hartley Park Villas, Tavistock Road, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Monday, January 31*st*, 1955

Joint Meeting with the Institution of Civil Engineers, at the South Wales Institute of Engineers, Park Place, Cardiff, at 6.30 p.m., when Dr. B. G. Neal, Ph.D. (Cantab.), A.M.I.C.E., will give a paper on " Simple Plastic Theory."

Wednesday, February 16*th*, 1955

Junior Members' Evening at the Mackworth Hotel, Swansea, at 6.30 p.m.

Tuesday, March 15*th*, 1955

At the South Wales Institute of Engineers, Park Place, Cardiff, at 6.30 p.m. Mr. A. C. Vivian, B.Sc., A.C.G.I., M.I.C.E., M.I.Struct.E., on " Kathodic Protection of Steel Structures."

Wednesday, March 30*th*, 1955

At the Mackworth Hotel, Swansea, at 6.30 p.m. Major A. F. Allen, M.I.C.E., on " Description of Neath By-Pass Constructional Work." (Illustrated with lantern slides.)

Hon. Secretary : K. J. Stewart, A.M.I.C.E., A.M.I.Struct.E., 15, Glanmor Road, Swansea.

WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, January 7*th*, 1955

At the University of Bristol Geology Lecture Theatre, at 6 p.m. Mr. H. G. Lakeman, A.C.G.I., B.Sc.(Eng.) London, A.M.I.C.E., will give a paper on " Recent Bridge Works in Bristol District."

Friday, February 4*th*, 1955

At the University of Bristol Geology Lecture Theatre, at 6 p.m. Mr. Clifford E. Saunders, M.I.Struct.E., will give a paper on " Some Effects of Prefabrication on Post-War Buildings."

Wednesday, February 16*th*, 1955

Annual Dinner at the Royal Hotel, Bristol.

Thursday, March 3*rd*, 1955

Combined Meeting with the Institution of Civil Engineers, in the Reception Room, University of Bristol, at 6 p.m., when Mr. W. J. Sivewright, M.A., A.M.I.C.E., and Mr. S. P. Whittington, B.Sc., A.M.I.C.E., will give a paper on " Reconstruction of Ground Floor of Cold Store, Avonmouth Docks."

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, January 19*th*, 1955

At the Great Northern Hotel, Leeds, at 6.30 p.m. Mr. F. A. Charman, B.Sc., A.M.I.C.E., will give a paper on " The Construction of The Woodhead New Tunnel."

Wednesday, February 16*th*, 1955

At the Great Northern Hotel, Leeds, at 6.30 p.m. Mr. F. N. Sparkes, M.Sc., M.I.C.E., will give a paper on " The Principles of Quality Control."

Wednesday, March 16*th*, 1955

To be arranged.

Friday, March 18*th*, 1955

Annual Dinner and Dance, at the Parkway Hotel, Bramhope, Leeds, at 7 p.m.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., P.O. Box 3306, Johannesburg, South Africa.

During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone : 34-1111, Ext. 257.

Natal Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co. Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., The Reinforcing Steel Co. Ltd., P.O. Box 2962, Cape Town.

The Late Mr. H. H. CLAPHAM

The Council deeply regret to record the death of Mr. Harry Hayes Clapham, B.Sc., M.I.Struct.E., which occurred on the 8th November at the age of 72.

Mr. Clapham had been a Member of the Institution since 1920. He was a Founder Member of the Lancashire and Cheshire Branch and served on the Committee of the Branch until a few years ago. He was Branch Chairman during the Charter Year, and in 1947 was elected a Vice-President of the Institution. In 1920 Mr. Clapham was appointed Head of the Building and Civil Engineering Department at the Royal Technical College, Salford, and continued in this capacity until his retirement in 1948.

His charm of manner and unfailing kindness endeared him to a wide circle of friends, and many structural engineers who passed through his hands as students will remember him with affection and gratitude.

Structural Prestressing*

By Felix J. Samuely, B.Sc. (Eng.) M.I.C.E., M.I.Struct.E.

1. Introduction

During the last two decades considerable attention has been given to prestressed concrete, and these two words have become almost inseparable. Actually, prestressing is a physical principle which has been used for thousands of years, although it was not recognised as such, and this physical principle is not confined to concrete.

Since prestressing has become a technical bye-word, it is being used more and more, but it is doubtful whether the circumstances in which prestressing may be usefully employed, and its limitations, are yet generally known. This paper is devoted to this problem, and the author shows the conclusions he has reached as to these limitations, particularly as far as structures are concerned. He will also give a number of examples of structures that have already been carried out, using prestressing. It must be said, however, right from the beginning, that we are still far from having exploited all the possibilities of prestressing, and there are several types of structure which will be mentioned in this paper although as yet there are no examples available, and of course there must be an enormous number of possibilities which will become known as prestressing develops and which could not even be hinted at yet.

2. Applications of prestressing

Apart from prestressed concrete, the only application of prestressing which has had a reasonable amount of publicity is prestressed steel (see Professor Magnel's paper printed in *The Structural Engineer* for November 1950, and also Mr. R. Sefton Jenkins' paper in February 1954). However, while there are a few examples available, prestressed steel is just one other application of the very much wider principle of prestressing. Long before the word " prestressing " was coined the principle of prestressing was employed, and it is used subconsciously even in some everyday objects. A few such historical or everyday items will be mentioned right at the beginning to make understanding of the following easier.

(a) Roman arches. The Romans countered the problem of arches tending to overthrow piers, by putting a large weight on to the pier ; in other words, to counteract the tension due to the over-turning bending moment they produced an initial compression stress in the pier, which is clearly tantamount to prestressing. This principle was even exploited architecturally, and gave rise to the typical Roman decoration of statues on piers (see Fig. 1).

(b) The common bolt relies on compressing the parts to be connected, between the head and the nut in order to get the necessary friction. This compression is obtained by tightening the nut and thus putting the bolt into tension.

Fig. 1.—Arch of Constantine, Rome

(c) The silk of an umbrella is a soft material which would hardly be considered strong enough to carry loads as a " slab." When the umbrella is opened this stresses the silk, which now becomes strong enough to carry a considerable load.

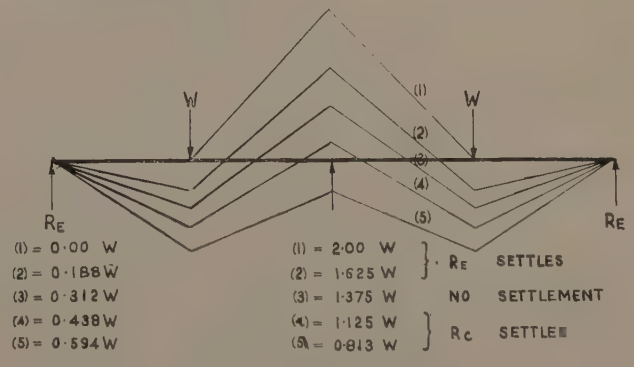


Fig. 2

In order to explain the principles of structural prestressing attention is drawn to the fact, which is very well known, that in all statically indeterminate structures the stresses may vary greatly if certain supports tend to yield. For instance, if a beam is supported at three points and due to the load the centre support settles more than the end supports, the negative bending moment at the support is reduced. It may even disappear or be replaced by a positive bending moment, while the positive bending moments between the supports are increased, and the greater the reduction

* *Paper to be read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 10th February, 1955, at 6 p.m.*

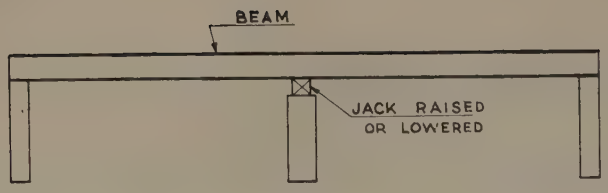


Fig. 3

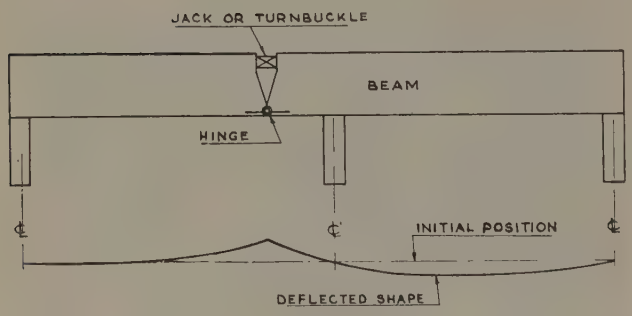


Fig. 4a

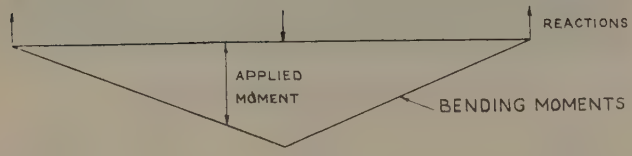


Fig. 4b

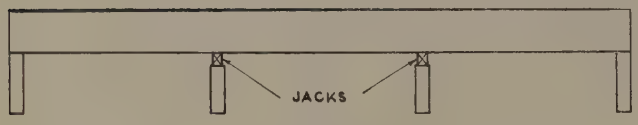


Fig. 5a

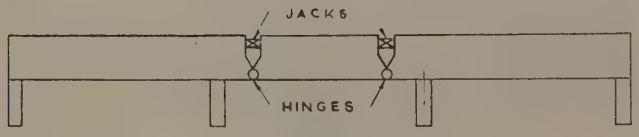


Fig. 5b

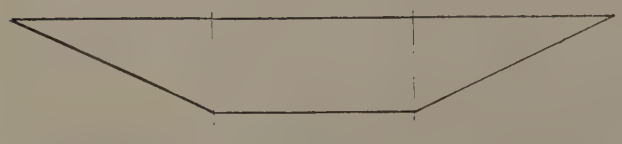


Fig. 5c

in the negative bending moment the greater is this increase. Conversely, if the outer supports are on uncertain ground and settle, the positive moments are reduced, the negative moment over the centre support is increased and may even spread over the whole beam (see Fig. 2). This possible variation of bending moments has always been considered a disadvantage of statically indeterminate structures where the ground is not firm enough to make settlement impossible.

However, if such settlement of supports is not left to chance but is controlled, the original disadvantage, namely the uncertainty of bending moments, can often be converted into an advantage. In Fig. 3 a beam on three supports is shown where the middle support consists of a jack that can be raised or lowered. By a reasoned use of the jack any one of the bending moments shown in Fig. 2 can be obtained, (1) or (2) by raising, and (4) or (5) by lowering the jack. If any of these bending moment diagrams is more suitable than any other, the designer can select the one he requires.

A similar effect can be produced if an internal moment is artificially introduced into the beam. Assume, for instance, that by a device which is diagrammatically represented in Fig. 4a two adjoining beams are rotated relative to each other, an artificial bending moment as shown in Fig. 4b is introduced, the magnitude of which depends on the angle of induced rotation at the hinge. This artificial moment can be superimposed upon the normal bending moment diagram *ad lib*, to produce the most suitable bending moment diagram. In this connection it is not material how the relevant diagram is obtained. That will be discussed later.

This simple example merely serves to illustrate a principle ; obviously in a system which has one redundancy one artificial movement like the jack in Fig. 3 or 4 may be introduced. For a beam which is statically indeterminate at two points, two jacks may be arranged, as shown in Fig. 5a or 5b of a beam on four supports. The bending moment is then as in Fig. 5c., and generally speaking the number of statically indeterminates defines the number of correcting jacks.

If the artificial introduction of stresses into a structure can be called " prestressing," obviously all this is prestressing, and it leads us to the general statement : **In any statically indeterminate structure it is possible, by deliberate prestressing, to produce any combination of bending moments and forces consistent with the laws of equilibrium.** If the structure is statically indeterminate at several points the prestressing has to be produced at the same number of points.

This appears to be a very significant state of affairs, and there will be many applications in which suitable constructions can be produced by prestressing. A complete, or even approximate, list of such applications cannot be given at this stage. These will emerge only in future practice, but here are a few examples :

(a) *Prestressed concrete*

A reinforced concrete beam is in itself a statically indeterminate structure. The concrete alone would be statically determinate, the tensile steel reinforcement being equivalent to an additional structural member which takes part of the forces out of the concrete beam. It is well known that before the advent of prestressed

concrete this was dealt with once and for all by the formula :

$$f_s = mf_c$$

where m = proportion of strain in concrete and steel, produced by equal forces

f_c = stress in the concrete at level of steel

f_s = stress in the steel,

i.e. as long as the bond between the two materials holds good the development under load of both materials must be equal. By giving additional stresses to steel and concrete before such bond takes place, i.e. stresses which do not obey the above formula, it is possible to change the relationship between steel and concrete stresses. In post-tensioned concrete a compressive stress is induced in the concrete while the steel is in tension, and in pre-tensioning a considerable tension force is induced in the steel without initially stressing the concrete at all. The release of the wires in pre-tensioned concrete is equivalent to introducing a compressive force into the member as a whole. This adds compressive stresses to the concrete and steel, which in fact obey the above formula, while the final relationship is different because of the initial stress in the steel due to the pre-tensioning. It may be noted that steel often is in tension while the concrete around it is in compression. In every instance, the relationship between stresses has been altered considerably to suit the qualities of the material, and it is well known today that by doing this it is possible to use both materials to their full advantage, and, for instance, high tensile steel may be used economically in reinforced concrete.

The greatest advantage arises out of the fact that the deformations are reduced considerably now that, owing to prestressing, the tensile stresses are transferred to the material that is capable of taking them. It will be noted that this reduction in deformation is a general characteristic of prestressing.

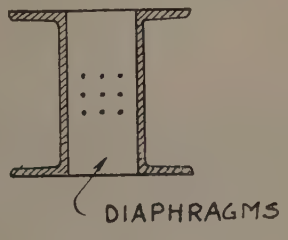


Fig. 6

(b) *Prestressed steel*

As a characteristic example of prestressed steel may be taken a tension member of a latticed girder consisting of two channels. Assume this section is as shown in Fig. 6 and that a number of wires are inserted between the webs of the channels, these wires being anchored to the channels at both ends and being prestressed. The channels are now in compression, and when external tensile forces are applied they have to overcome this compression before they can produce tension in the member. As the modulus of elasticity of the wires and the channel is approximately equal, the tension stresses induced in the wires by external forces are approximately the same as those in the channel, but while they add to the original stress in the wires they are deducted from the original compressive stress in the channels. The stresses in the wire are therefore much higher than those in the channel, and by a reasonable choice of section can be in the same proportion as the permissible stresses. Prestressing therefore again allows the use of two materials of different stress capacity side by side, although there is no difference in the modulus of elasticity. The total weight used with prestressing is, in every instance, smaller than the one without prestressing, and the proportional saving is given in the following formula :—

$$\frac{c}{c+t}$$

If the wires in such a prestressed steel member were connected at the ends only, the compressive force due to prestressing would cause buckling in the channels. However, if the wires and the channel are connected at intervals the distance between any two connections may be taken as the buckling length and such connections can, of course, be sufficiently near together so that the buckling is of no importance. It must be realised that the compressive stress c can often be taken very much higher than the tensile stress t. It is reasonable to have a much smaller factor of safety for the compressive stress than for the tensile stress because it is certain that the member will never get any higher compressive stress than the one induced by prestressing, while the load will immediately reduce the compressive stress.

The proportion of the quantity of mild steel and wires required varies with the ratio K of the working stresses in the high tensile wire to that in the mild steel member, and Table 1 shows the amount of mild steel and wires expressed as a percentage of the weight of a non-prestressed tension member, based on the assumption that $c = 1.25\,t$.

Table 1

$K = \frac{\text{Stress in wire}}{\text{tensile stress in mild steel}}$	4	5	6	7	8	9
Amount of mild steel %	25.9	30.6	33.4	35.3	36.6	37.6
Amount of H.T. steel %	18.6	13.9	11.1	9.2	7.9	6.9

The use of tubes or other closed sections has been suggested for prestressing steel, but in actual fact there will always be the problem of stabilising the tube against the wires at intervals, which is not easy, while if the wires are in the open, as shown in Fig. 6, diaphragms at certain centres can easily be arranged to hold the wire in position. Emphasis should be laid again on the fact that the controlled allocation of stresses would be impossible without prestressing, unless one of the two materials is stressed so highly that the elastic limit is exceeded and plastic flow equalises the stresses—see also the later paragraph on plasticity.

A system of prestressing applicable to high tensile steel joists, for which the deflection under working stress would be excessive, has been developed in Belgium. The joists are flexed in pairs against each other, and concrete cast around the tensile flanges. When this concrete has hardened the jacks are removed, applying a moment of the opposite sign to the now partly encased beam. Compressive stresses are set up in the concrete, but a residual stress remains in the encased steel. After erection the rest of the beam is normally

encased in concrete so as to provide an adequate compression flange. When the beam is loaded to the permissible stress in the tensile flange the deflection is very small, and by suitably adjusting the prestress cracking of the concrete casing can be prevented.

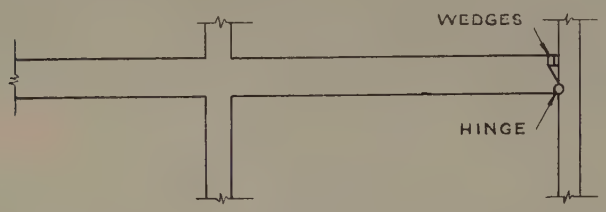


Fig. 7

(c) *The continuous beam or slab*

The bending moment diagram obtained in a continuous beam is often not satisfactory from the designer's point of view, even if the bending moments are corrected to allow for a certain amount of plasticity. Very often, particularly with steel joists, it is useful to have the positive and negative bending moments equal. It happens, on the other hand, quite frequently, that it would be more satisfactory either to have at the support very large bending moments or no bending moments at all. If, in the latter case, there were a number of beams calculated as non-continuous, particularly with reinforced concrete, the result would be cracks over the supports. It is, however, possible to introduce artificially a bending moment over the supports by a one-sided jack as shown in Fig. 7, and if concrete is inserted between the beams after the jacking operation is finished and the jack removed, an additional bending moment diagram is produced which can be of such magnitude that when the load is applied the final moment at the supports is zero. We have now the equivalent of a series of non-continuous beams without losing the beneficial effect of continuity and without danger of cracks at the supports. Another method of introducing a negative bending moment at the supports is by means of turnbuckles as shown in Fig. 8. If the real load is then applied, the final negative bending moment will be reduced by the amount of the bending moment introduced by the turnbuckles. Depending on external circumstances—for instance, the relative depth of beams at the support and in the centre, whether the beams are upstanding or downstanding—the necessity of reducing deflection, etc., will often make such control desirable.

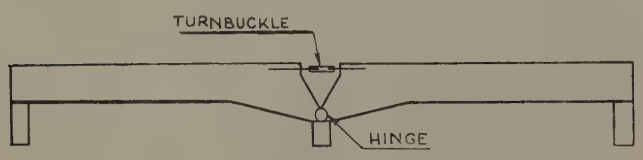


Fig. 8

This method of redistributing moments was used in the construction of continuous steel beams of three 40 ft. spans for a warehouse roof in Fairbanks, Alaska. To equalise the moments in the end spans and at the supports the beam was lowered $2\frac{1}{2}$ per cent. at the internal columns. By doing this the maximum moment was reduced by 15 per cent. and a saving of 8 per cent. in the weight of steel was effected.

The Luzancy bridge in France also incorporated this method of prestressing. To obtain the shallow construction depth at mid span which was necessary to give the required clearance above the water level, a fixed ended arch bridge of low rise was chosen. Permanent jacks were inserted between the abutments and the lower chord of the triangulated concrete frame at the ends of the bridge. There was a tensile connection between the top of the bridge and the abutment, which point therefore became the hinge when jacking commenced. By suitable adjustment of the force in these jacks the required amount of horizontal thrust could be provided, and if the abutment moved, reducing this thrust, it could be restored to its initial value by pumping up the jacks once more. As this jacking force was applied eccentrically it was equivalent to a fixed end moment.

(d) *The connection of beams to columns*

Eccentricity or rigidity of connection between columns and beams often introduce bending moments into columns and at the end of beams. These moments usually reduce the bending moments in the centre of the beam. This is an advantage for the beam, but the bending moments in the columns are often very awkward to deal with. Particularly in the case of long columns, it would often be useful to reduce the bending moments in the columns although this would increase the stress in the beams. This can be done by introducing jacks or wedges, in a similar way as in the previous example, between the columns and the beam (see Fig. 7).

Where beams and columns are connected to act as rigid frames it is perfectly possible to construct them, in the first instance, so that they do not fit properly at the supports, and to introduce a horizontal thrust at both ends, either inward or outward, to make them fit after a certain arbitrary force is applied. This means that an actual positive or negative bending moment is superimposed on the whole frame, and that correspondingly the bending moments are altered to suit the designer.

(e) *Trussed beams*

A very economical combination of steel, timber or precast concrete with a steel tie is the trussed beam. However, if such trussed beam is comparatively shallow (Fig. 9), the distribution of stresses is often not very satisfactory, particularly with high tensile steel when the area of steel is very small and the moment of inertia of the top chord large.

The upper chord then takes too much of the bending moment and the truss as a whole too little. By tensioning the steel, upward forces are induced in the struts as shown in Fig. 10 which tend to deflect the chord upwards. These upward forces act against the external loads, reducing the moment in the top beam, so that while the stresses due to direct forces are increased, the maximum stress in the upper chord is decreased and by controlling the amount of tensioning it is possible to :

(i) reduce the deflection of the upper chord under load or even produce a camber in it ;

(ii) distribute the forces so that both the material in the upper chord and the tie can be used to their respective capacities.

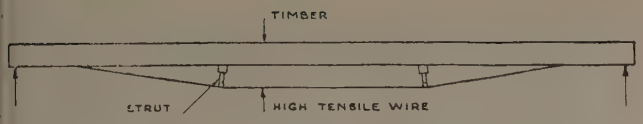


Fig. 9.—Trussed Beam elevation

Example of a trussed beam calculation

For a particular span to depth ratio the reduction of the bending moment caused by the truss action depends principally on the relative values of the flexural rigidity of the top beam and the extensibility of the tie. Referring to Fig. 9 the moment carried by the truss action for a uniformly distributed load is as follows :—

To determine the distribution of forces and moments the truss is made statically determinate by cutting the tie. Then using the influence coefficient method with the tie force X as the redundancy, the virtual work equation for a total distributed load $3W$ applied to the top chord (where the struts are assumed incompressible) is:—

$$X1\left\{\frac{3L+\frac{D^2}{L}}{E_S A_S}+\frac{3L}{E_T A_T}+\frac{2D^2L}{3E_T I_T}+\frac{D^2L}{E_T I_T}\right\}$$

$$=\frac{9}{8}\frac{WL}{E_T I_T}\left\{\frac{LD}{3}\left(\frac{5}{9}\times\frac{1}{2}\times 4+\frac{8}{9}\times 1\times 1\times 1\right)\right\}$$

$$\text{i.e. } X1\left\{\frac{3L+\frac{D^2}{L}}{E_S A_S}+\frac{3L}{E_T A_T}+\frac{5}{3}\frac{D^2L}{E_T I_T}\right\}=\frac{11}{6}\frac{WL^2D}{E_T I_T}$$

$$X1=\frac{\frac{11}{6}\frac{WL^2D}{E_T I_T}}{\frac{3L+\frac{D^2}{L}}{E_S A_S}+\frac{3L}{E_T A_T}+\frac{5}{3}\frac{D^2L}{E_T I_T}}$$

$$=\frac{\frac{11}{6}W\frac{L}{D}}{\frac{E_T I_T}{E_S A_S D^2}\left(3+\frac{D^2}{L^2}\right)+\frac{3I_T}{D^2 A_T}+\frac{5}{3}} \quad (1)$$

The total moment at mid span $=\frac{9}{8}WL$ (2)

Where W = total load on length L

$E_T I_T$ = flexural rigidity of top chord

$$=1.5\times 10^6\times\left[\frac{5}{12}\times 8^3\right]$$

$$=320\times 10^6 \text{ lbs/in.}^2$$

E_S = modulus of elasticity of tie

$$=21\times 10^6 \text{ lbs/in.}^2$$

A_S = area of tie 0.167 in.^{2}

A_T = area of top chord $= 8\times 5 = 40$ in.^{2}

E_T 2 modulus of elasticity of top chord

$$=1.5\times 10^6 \text{ lbs/in.}^2$$

$3L$ = span of truss = 18 ft. 0 in.

D = 12 in. = depth of truss.

The moment taken by the beam at mid span is (2) — (1)

and for a completely inextensible tie

$$+\left(\frac{9}{8}-\frac{33}{30}\right)WL=\frac{WL}{40}$$

For an extensible tie this moment increases and the larger the value of $\frac{E_T I_T}{E_S A_S D^2}$ the larger the proportion of the moment carried by the beam. For a timber or concrete top chord and a high tensile tie of the proportions shown in Fig. 9, the truss action takes 44 per cent. and the top chord 56 per cent. of the total moment. If the dead load and the live load are equal and the tie is prestressed when the full dead load is acting, so that the top chord has a hogging moment $=\frac{1.69}{8}WL$ at the post which will not cause the allowable stress to be exceeded, the percentage of the total moment carried by the truss action under full live load is increased to 85 per cent. of $\frac{9}{8}WL$. Consequently the direct force in

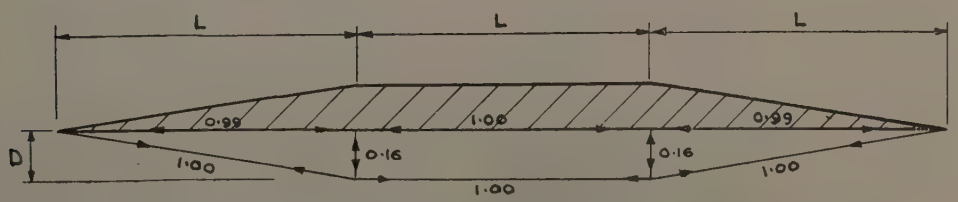


Fig. 10.—Forces and moments in trussed beam due to unit prestress in tie

the top chord is increased in the ratio of $\frac{85}{44} = 1.93$, and

the moment at mid span which is $\frac{1.35\ WL}{8}$ is decreased

in the ratio $\frac{15}{56} = 0.27$, as compared with a non-prestressed tie.

The maximum stress in the top chord of the prestressed truss is as follows :—

$$\frac{(9—1.35)\ WL}{D \times 8 \times A_T} + \frac{1.35\ WL}{8 \times I_T} \times 4''$$
$$= 0.143W + 0.228W = 0.371W$$

compared with

$$\frac{9 \times 0.44WL}{D \times 8 \times A_T} + \frac{05^6 \times 9WL \times 4''}{8 \times I_T}$$
$$= 0.074W + 0.85W = 0.924W$$

if the tie is not prestressed.

Hence for the same dimension of truss prestressing increases the maximum load to 2.5 times that of the non-prestressed truss.

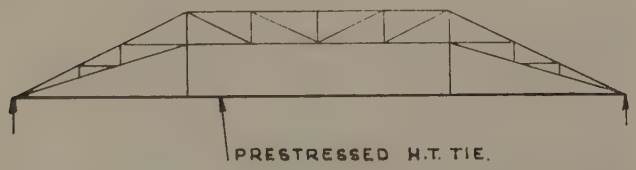


Fig. 11a.—Inverted trussed beam

If the upper chord is made from precast concrete, or indeed from steel of different quality from the tensile member (usually inferior quality) the same considerations apply. The construction of a bowstring girder or rigid frame with a high tensile steel tie (Figs. 11a and 11b) is a very similar example. In this case, particularly when the height of the beam is small compared with the length, the extension of the tie is very great, and in order to retain the effect of a two-hinged frame, stresses in the tie must normally be kept very low, making this tie very extensive. The alternative of prestressing the tie allows any initial deformation to be reduced to a preconceived amount, or even to zero, and in this way the full effect of the two-hinged construction can be achieved (see also Mr. Sefton Jenkins' paper of February 1954 previously referred to, which dealt with a specific example of this type). Prestressing of this type should be of particular importance with two hinged frames or arches where the ground cannot be relied on to take

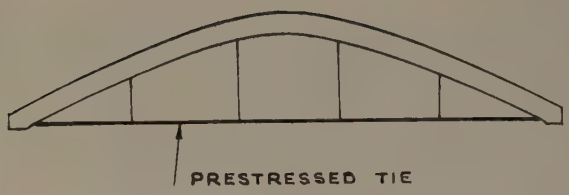


Fig. 11b.—Bowstring girder

horizontal forces without rather indefinite deformations. The use of an ordinary tie member in such cases usually leads to troublesome cracks near the foundations, but a prestressed tie member can be relied on to have a very reduced elongation after tensioning.

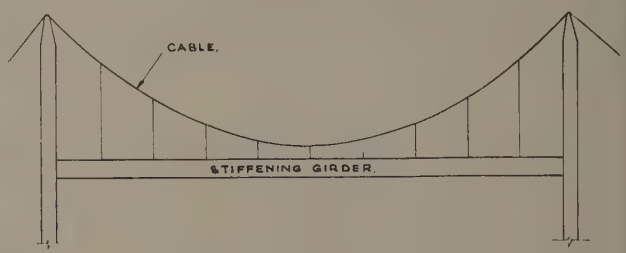


Fig. 12a.—Suspension bridge

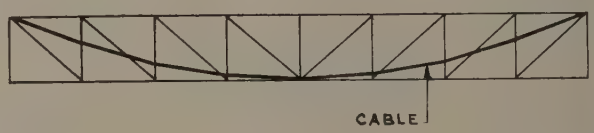


Fig. 12b

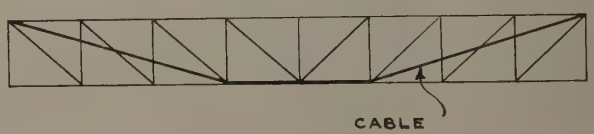


Fig. 12c

(f) *Suspension girder*

It is a well-known fact that the cheapest way to carry a uniformly distributed load is by means of a cable hanging in the approximate shape of a parabola. This fact has been used in suspension bridges, and in these structures the cable is stiffened by a girder which takes the difference between the actual load and the uniformly distributed load carried by the cable. Most suspenison bridges are arranged so that the sag of the cable line is very much larger than the depth of the girder, and in this way it is ensured that the girder really does not do considerably more work than the minimum necessary

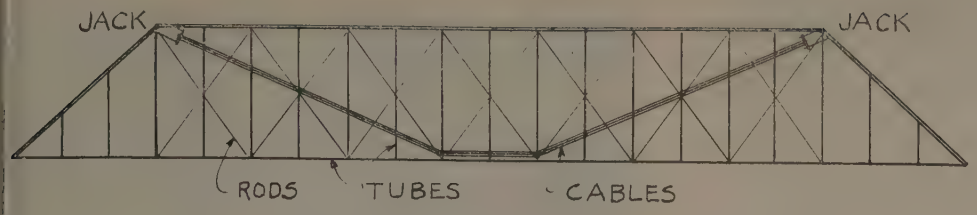


Fig. 13a

to distribute the load on to the cable (Fig. 12a). If, on the other hand, such a bridge were constructed in accordance with Figs. 12b or 12c, i.e. with a cable the sag of which is approximately the same as the depth of the girder, most of the load would be carried by the girder and the cable would be almost superfluous.

By means of prestressing it is, however, possible to change this and to make effective a cable within the depth of the girder ; see Fig. 13a. By doing this the principle of a suspension bridge can be applied to an ordinary parallel girder, and a considerable saving, both in weight and cost, achieved. Such a construction is shown in Fig. 13b of the roof of a workshop for a School in Wigan, and in this case, small jacks have been built into the verticals of the trusses which, when tightened, raise the latticed girder against the cable. By doing so the stresses are taken out of the lower chord and the diagonals, and are introduced into the cable which is stressed to a much higher degree. The upper chord is not appreciably affected by the jacking, except in end bays of the truss, where the force is increased to about the same value as that at midspan. It is, of course, the most correct method to have jacks in every vertical, but as Fig. 13a shows it is quite sufficient to have two jacks altogether, the cable then having two sharp kinks instead of a parabolic form. It might be said that this application was not the best possible example since, as the latticed girders formed the inclined sides of a hipped roof, the cables could not be extended to the support and were therefore not 100 per cent. effective (see Appendix 1).

(g) *Prestressing a structure as a whole*

Very frequently it is advantageous to reduce to a minimum the number of members which have to be designed for compressive stresses. This is similar, but opposite, to the attempt made in prestressed concrete to reduce the tension members and have compressive members only. For all structures which have very long members it is advisable to have only tension members, or if this is not possible, as many tension members as can be conveniently arranged. This can frequently be achieved by prestressing, and the principle is that if a member has sufficient tension beforehand it can then be stressed in compression afterwards without producing actual compression, as the compression stress applied merely reduces the original tension.

The following three examples of work carried out give some idea of what can be done in this respect, but in actual fact prestressing will be even more useful in conjunction with space construction than it is with simple structures ; however, very little is being done in this field at present, and no example is available.

(i) It is quite common to design wind girders with the cross diagonals in such a way that only the tension diagonal is in action, while the compression diagonal does not really work because of its great slenderness. By doing so it is possible to avoid wastage of material in long compression diagonals. However, where this construction is used the girder must, of course, have a considerable initial deflection until it works in the way it is designed. To begin with, of two cross diagonals in a latticed girder one will take as much stress as the

Fig. 13b

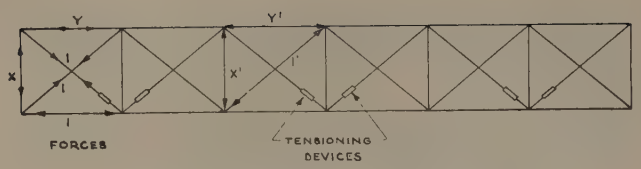


Fig. 14.—Wind girder with X bracing

other if there is any shear, and only after the compression diagonal has deflected sideways sufficiently and shown that it is not capable of taking such compression will the tension member take the full shear. It would therefore be rather disastrous to use such cross diagonals where deformation has to be reduced to a minimum, but in such cases a construction as shown in Fig. 14 may be used with prestressed diagonals. For instance, if in a truss a shear S is to be taken, which may act in either direction, the two diagonals should each be prestressed to an amount corresponding to half the shear. When the actual shear S is applied, this will cause tension in one of the diagonals, stressing it up completely, and compression in the other diagonal, which will bring the prestress down to zero, but the deformation in this case will be purely elastic and there is no initial slack to be taken up. It is possible to use bars or cables for the diagonals, and it is particularly interesting to note that only one of the two diagonals has actually to be artificially tensioned (see Fig. 14), while the other one is

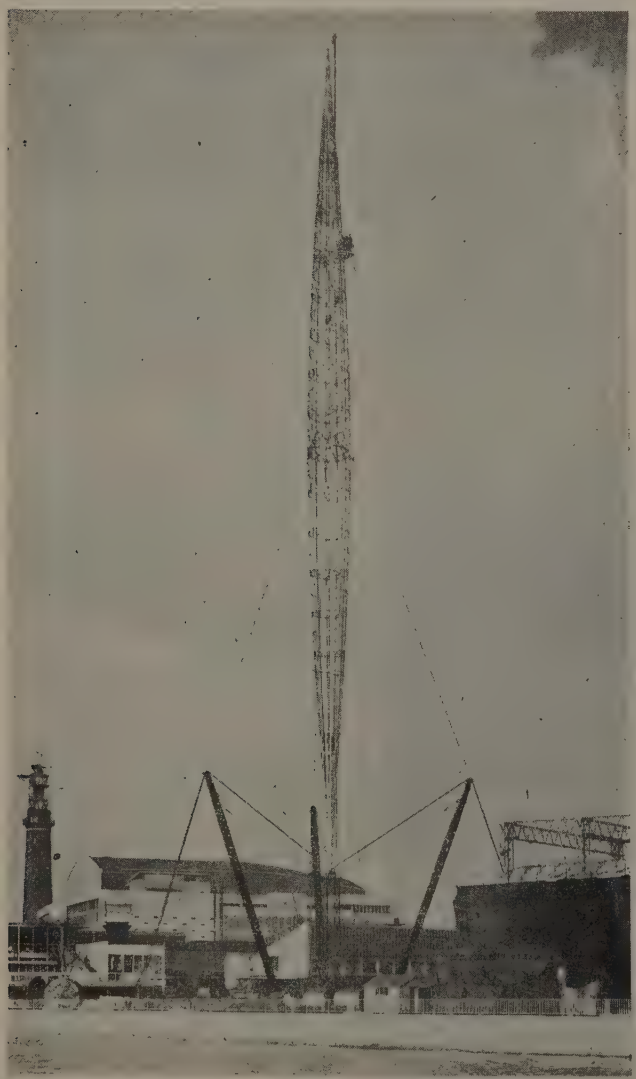


Fig. 15

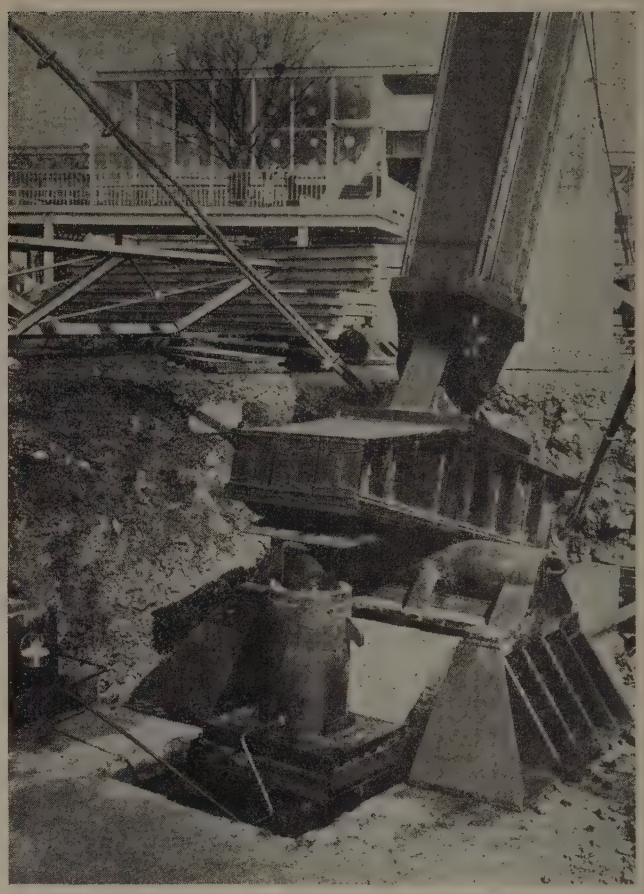


Fig. 16

stressed automatically. As the second diagonal lengthens during tensioning by an amount that can be calculated beforehand, it would be best to give the construction an initial set so that after tensioning it has the correct shape.

(ii) Another example is The Skylon (see Part 1 of the Proceedings of the Institution of Civil Engineers, July 1952).

This consisted of a mast balanced in the air by cables only, which were anchored in the ground and held in position by means of pylons (a)—see Fig. 15. There were six cables altogether, three going from the ground, to points x, y, z, at about the middle of the Skylon. (b) and three other cables going from point x to y, yo to z and z to x respectively via the base of the Skylon. For instance, one of these cables ran from x to the top of pylon 1, then from there to the base of the Skylon, then to the top of pylon 2 and down to point y. The two others were similarly arranged. If such cables had only been arranged with the usual tautness, this would have resulted in a very slack system, allowing considerable movement of the Skylon itself, and the dynamic forces set up under wind load would have been such that enormously strong cables would have been necessary to deal with them. By prestressing these cables the whole system became taut and the movements under wind were restricted to about $1\frac{1}{2}$ per cent. of what they would otherwise have been. This prestressing was done by jacking up the pylons after the Skylon was erected to a point just above b. It will be readily noted from Fig. 16 that due to raising of the pylons, forces were introduced in the cables and the actual prestresses were 99 tons in the lower and 55 tons in the upper cables, producing 156 tons compression in each pylon and 154 tons in the Skylon itself.

It has been remarked that by prestressing, certain forces can be transferred from one structural member to another that can better take it. Considerable progress has been made, at least in the laboratory, during the last few years in making use of the one thing that is available free of charge, namely, the earth itself, and while such examples are at the moment still models, one of these days they will probably become a reality. The author has, for instance, seen, at an American University, an interesting model of an arch that consisted entirely of wires held by stiff diaphragms, and it was capable of carrying a reasonable amount of load after being prestressed against its base.

Fig. 17

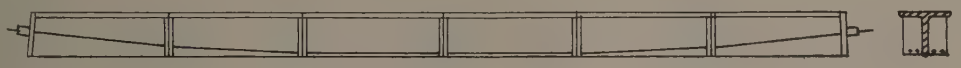


Fig. 18

(iii) A continuous suspension bridge of five spans was recently constructed in South America, in which cables were used for all the main structural members. The conventional stiffening girder used to transfer live load from the deck to the cable was replaced by a truss consisting entirely of prestressed cables. The top chord was the main suspension cable, and the web members were crossed diagonals which also performed the function of hangers carrying the road slab and were thus prestressed by the loads on the deck. The lower chord of the truss consisted of steel cables which were prestressed by jacking and were anchored to the ends of the concrete road slab. It was necessary to pre-tension these cables since they would have to carry compressive stresses at the towers. The concrete slab was allowed to slide freely over the stringers, cross beams and main longitudinal girders, which latter were supported by a hanger pin-jointed to a saddle round which the diagonal cables passed. The cables forming the diagonals were rigidly attached to the suspension cable, but at their lower end passed under this saddle, and during construction could slide freely round it and thus have equal tension in them. The lower chord cables, after pretensioning, were clamped to the saddles. To ensure that there was sufficient prestress in the diagonals also so that under all conditions of loading some did not become slack, one third of the live load was placed on the bridge before clamping the hangers to the lower chord of the truss. On the removal of this load an additional tension was caused in those diagonals which would otherwise have become slack under full live load. Once the diagonals were clamped to the lower chord, the whole structure behaved as a lattice girder continuous over five spans. Consequently, the removal of the one third total live load after construction was equivalent to prestressing the entire structure, so that every member received a stress of the opposite sign to that which it would receive under working load. As a net result the truss action carried two-thirds of the live load.

As the lower chord was anchored against the deck slab only at its ends and not at the tops of the towers, the deflection in the deck slab was much less than it would have been in the case of a single span bridge.

(h) *Composition of structures from standard units*

There are a great number of examples in prestressed concrete where a beam is composed of small units set side by side and compressed together by a cable or cables going the whole length of the beam. These units act together as one beam because, provided the shear is taken somewhere, there is no need to take any other stresses in the tension zone except those which are taken by the connecting cables. In the compression zone, on the other hand, a number of units leaning against each other can transmit the stresses easily. This, of course, can be done just as well with units other than reinforced concrete.

To take the famous example which Freyssinet gave as an explanation for such a beam, one could take a number of books, press them together by hand from each end and they would be able to carry not only themselves but other loads as well. While books may not be suitable material for structures, this method becomes very suitable when any standard units are to be assembled. It would be perfectly feasible, for instance, to join two joists (see Fig. 17) by prestressing, i.e. by putting two bars from one end to the other and tensioning them. As the tension zone of the joists does not do any work in the final state of stress, this arrangement would probably be rather uneconomical. It

Fig. 19

becomes very much more economical with, for instance, a series of tees connected by nothing else but two long bolts or cables (see Fig. 18), and it becomes particularly useful where standard units are concerned which cannot easily be connected in tension. Fig. 19 shows a number of standard pyramids erected to form a roof construction, and there are merely cables inserted which are prestressed to the full amount of tension that could occur in the lower chord of the construction. In this case, and in similar applications, the advantage of the prestressing is not so much in the material as in the easy method of assembly by merely " stringing " up the units. (This type of pyramid roof has been developed under the name of " Unitectum.")

3. Pre-tensioning and post-tensioning

The great problem with every prestressed structure is the fact that to begin with during prestressing certain stresses, for which allowance must be made, are set up. For instance, in example (d), while it can be arranged that there are no bending stresses in the column when it is fully loaded, there would be bending stresses due to prestressing if this is done before any load comes on to the column. These bending stresses may be rather more than the column can stand if it is designed for a centre load only, and in such cases a compromise may have to be made, prestressing to such a degree that the most economical column is obtained. Generally speaking, where materials are used which can take bending equally in both directions, it is always possible to halve the final bending moment by prestressing to one half the bending moment in the opposite direction. The material would have to stand up to half the bending moment before the load comes on, and one half after. If it is permissible to have a smaller factor of safety before the load is applied (see also the reference to prestressed steel) then further savings can be made, and for instance, a joist may be prestressed upward to take .6 of the final bending moment so that the final stress is proportional to that caused by 40 per cent. of the bending moment only.

Alternatively, it will in many cases be possible to prestress structures only after a certain part of the load, i.e. the full dead load or part of it, has been applied (post-tensioning). In these cases bending stresses occasioned in the first instance would be very much smaller, then made to disappear by the prestressing, which would usually produce stresses in the opposite direction, and these in their turn would disappear when the full load comes on. The ideal solution would be of course if the prestressing could be applied gradually at the same time as the load. This of course is impractical, but the calculation given as Appendix 2 shows how, for instance, in the case of a trussed beam, prestressing in two stages can be usefully applied.

4. Methods of prestressing

There are, generally speaking, two main means of prestressing, i.e. by tension or by compression. At the moment tension is generally produced by means of wire and compression by means of jacks, but it is quite likely that as experience with prestressing increases other ways of dealing with this problem will be found.

It has been explained before that prestressing means to put a preconceived combination of stresses into a structure which in itself must be in equilibrium, so that in fact the total sum of stresses is always zero in every direction and has a zero moment about any point. If a certain amount of these stresses is introduced the remainder follows automatically (see also the example of the single panel of a latticed construction where the prestressing of one diagonal automatically introduces stress in the crossing diagonal). When designing the prestressing operation for a structure it is therefore essential to weigh the various possibilities and to induce stresses where they are most easily induced and not necessarily those which are directly most important. In prestressed concrete, for instance, it is usual today to introduce the compression in the concrete by tensioning the steel, which is simpler, and not by compressing the concrete, but it is possible to imitate the Roman construction mentioned earlier and to produce the compression in a pier by introducing an arbitrary weight.

Generally speaking, forces are produced by geometric variation of points, i.e. if two points are brought nearer together compression is induced; by moving them apart, tension; and similarly bending and torsion are induced by rotations relative to certain axes, keeping the distance of points from those axes constant. It is therefore possible to choose between the following four methods of prestressing :

(i) By increasing the distance between two points (tensioning) ;

(ii) by reducing the distance between two points (compressioning) ;

(iii) by turning one cross section of a member in relation to another (torsioning) ;

(iv) by inclining a section of a member against another (off-lining).

There is a tendency today to give a preference to jacking for prestressing. Strictly speaking, by jacking, compression is introduced, and jacking in its true sense was used, for instance, for the Skylon, but the practical advantages of this method are so great that they are used for tensioning as well, i.e. the distance between two points A and B is reduced, but if B remains static while A moves nearer to it the distance from A to a point C on the extended line BA is automatically increased.

The various jacks which are on the market are well-known, and where tension is to be induced the dimension of such jacks is of relatively little importance (although, of course, the special consideration of surroundings may make the application of a large jack difficult). However, where compression is used, two members may be very close together to start with, and in this case it is imperative to reduce dimensions of the jack to a minimum. For such purposes a flat jack has been devised.

5. Plasticity and Prestressing

It must be realised that prestressing is in competition with plasticity, and that in principle similar results can be achieved by prestressing as by relying on plasticity. The most important differences are as follows :

Considering plasticity, it is assumed that a bending moment distribution can be achieved different from the one in accordance with the theory of elasticity, because most materials tend to be plastic, i.e. are capable of deforming considerably under high stresses. This plastic redistribution of stresses is therefore usually

achieved by additional deformation, which sometimes can be quite excessive. The redistribution due to prestressing can usually be achieved with a reduction of deformation, and particularly if post-tensioning is used, by having a reverse deformation without the live load and a relatively small deformation when the live load is applied.

Not all materials are really plastic—for instance, high tensile steel and other materials where the strain over stress ratio does not increase considerably before failure do not lend themselves too well to such methods, while they would respond to prestressing. Even materials which are suitable for plastic methods of calculation are not really useful where there is a great variation of stresses, and particularly where there are reversible bending moments or danger of fatigue.

On the other hand, prestressing usually produces reverse bending moments to the final ones at some stage or other, and where this complicates matters the point has to be taken into consideration, as shown in the section on " Pre-tensioning and post-tensioning." This can largely be overcome by having the stressing done in a number of stages, but if this number becomes too large other complications may arise.

In all cases where the deformation has to be kept within certain limits, and where rigidity and stiffness are important, better results will be obtained by prestressing than by relying on plasticity, and often even greater economy.

APPENDIX I

Steel Girder with Prestressing Cable

The roof trusses are arranged in the planes of the roof. The loads on the roof are carried by trussed rafters to the ridge and valley of the roof. At those points they can be resolved into the planes of the roof, and hence the load acting on each truss can be ascertained.

A.—Below is set out the calculation showing the force in each member of the truss and in the cable, if the cable is not prestressed.

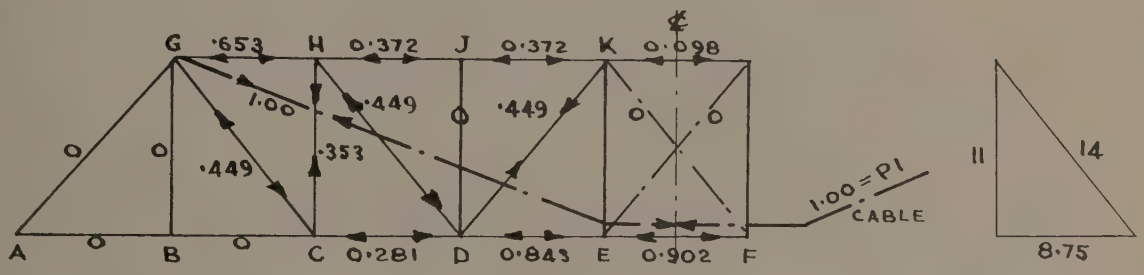


Diagram P.1. Elevation of half-truss. Forces due to unit force in cable

Only those diagonals which are in tension in the prestressed roof are considered and it is assumed, for analysis purposes, that they can resist compression, although under actual load conditions they do not.

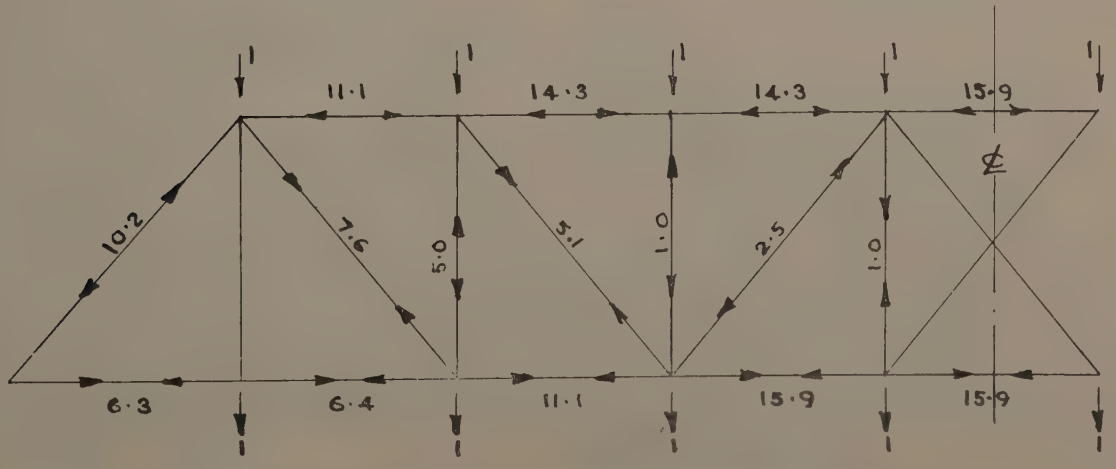


Diagram P_0. Forces in truss due to unit load at each panel point, with cable cut

Member	$\frac{l}{AE}$	P_1^2	P_1P_o	$\frac{l}{AE}P_1^2$	$\frac{l}{AE}P_1P_o$
GH	0.91	0.43	7.3	0.39	6.6
HJ	0.91	0.14	5.3	0.12	4.8
JK	0.91	0.14	5.3	0.12	4.8
$\frac{1}{2}KL$	0.46	0.01	1.6	0.01	0.7
CD	1.99	0.08	—3.1	0.15	—6.2
DE	1.99	0.71	—13.4	1.41	—26.6
$\frac{1}{2}EF$	0.99	0.81	—14.3	0.80	—14.2
CH	3.67	0.13	—1.8	0.46	—6.5
EK	3.67	0.13	—0.4	0.46	—1.2
6*C*	7.13	0.20	—3.4	1.44	—24.5
4*D*	12.75	0.20	—2.3	2.58	—29.1
DK	18.1	0.20	—1.2	3.65	—20.8
Cable	89.7	1.0	0	89.7	0
	Total ..			+101.32	—112.2

$\therefore$ $101.32\ P_1 = 112.2$

$P_1 = 1.11$

Max. Force in bottom chord = 14.9

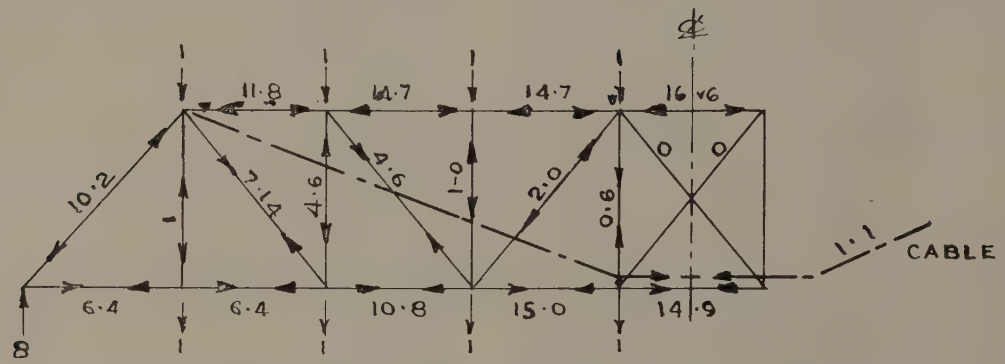
Max. Force in top chord = 16.0

Moment in tie and top chord = $1.11 \times 0.9 = 1.00$

Moment in bottom chord = $14.89 \times 1 = 14.9$

Percentage taken by tie = 6.3%

Then the forces in the truss are :



Not prestressed. Girder forces Dead and Live loads

B.—When the cable is prestressed the calculation is as follows :

The live load formed 40 per cent. of the total load.

If the tie is prestressed to take the dead load, i.e. 60 per cent. of the total moment,

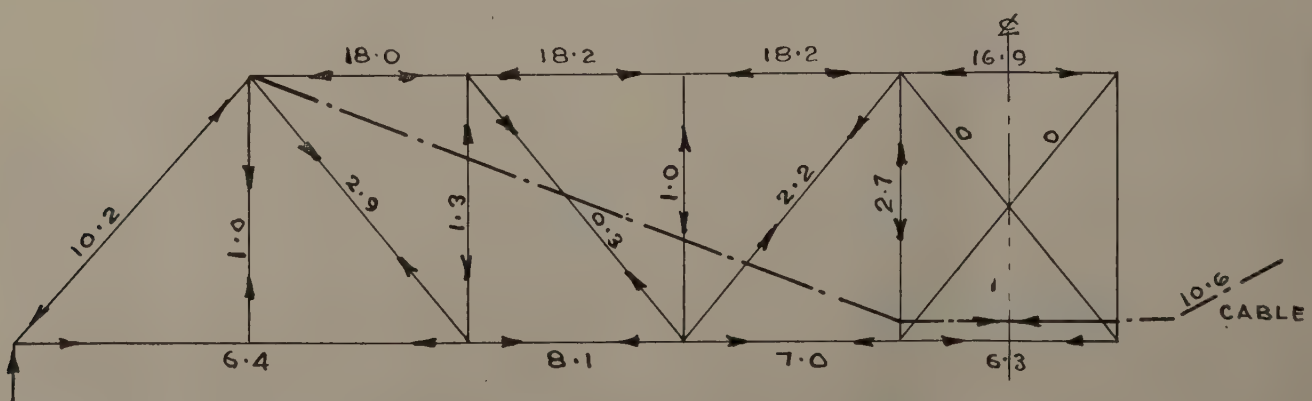
Total Moment $= 16 \times 1 - 1.1 \times 0.1$

$= 16 - 0.1 = 15.9$

Dead Load Moment $= 0.6 \times 15.9 = 9.54$

Force in cable $= \frac{0.6}{0.9} \times 15.9 = 10.6$

Then the forces in the truss are :



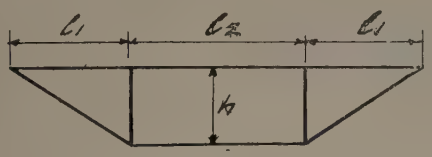
Prestressed. Girder forces Dead + Live loads

APPENDIX II

Precast Concrete Trussed Beam

(Prestressing in two stages)

In order to keep the moments in the beam to a minimum the prestressing is done in two stages



$$2l_1 + l_2 = l$$

Properties of Sections

$E_c = 5 \times 10^6$ lbs/in.^{2} (before creep)

$= 2 \times 10^6$ (after creep)

$$I_c = \frac{4 \times 6^3}{12} = 72 \text{ in.}^4 = \frac{1}{2 \times 144} \text{ ft.}^4$$

$$A_c = 4 \times 6 = 24 \text{ in.}^2 = \frac{1}{6} \text{ ft.}^2$$

$$\frac{A_c}{I_c} = \frac{1}{6} \times 2 \times 144 = 48 \frac{1}{\text{ft.}^2}$$

$E_s = 17 \times 10^6$ lbs/in.^{2} In working range

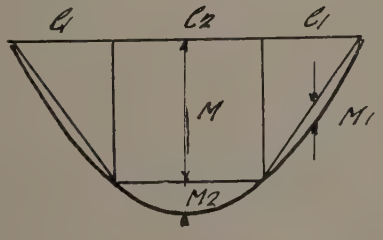
$$A_s = 5 \times .0244 = .122 \text{ in.}^2 = \frac{.122}{144} \text{ ft.}^2$$

$$\frac{A_s}{I_c} = \frac{.122}{144} \times 144 \times 2 = 0.244 \frac{1}{\text{ft.}^2}$$

Force T in tie is determined by the δik method.

This gives

$$T = \frac{\delta oa}{\delta aa}$$



For the case

$l = 25$ ft. $l_1 = 7.67'$ $l_2 = 9.67'$ $h = 14''$

$$M_1 = \frac{wl_1^2}{8} = 7.35\,w \text{ lbs/ft.} \quad . \quad . \quad . \quad . \quad (1)$$

$$M + M_2 = \frac{wl^2}{8} = 78.1\,w \text{ lbs/ft.} \quad . \quad . \quad . \quad (2)$$

$$M_2 = \frac{wl_2^2}{8} = 11.7\,w \text{ lbs/ft.} \quad . \quad . \quad . \quad . \quad (3)$$

$$M = (2) - (3) = 66.4\,w \text{ lbs/ft.} \quad . \quad . \quad (4)$$

$$\delta aa = h^2\left(\frac{2l_1}{3} + l_2\right) + l\left(\frac{I_c}{A_c} + \frac{E_c\,I_c}{A_s\,E_s}\right)$$

$$= h^2\left(\frac{2\times767}{3} + 9.67\right) + 25\left(\frac{1}{48} + \frac{E_c}{0.244\,E_s}\right)$$

$$= 14.78\,h^2 + 25\left(\frac{1}{48} + \frac{E_c}{0.244\,E_s}\right)$$

$$\delta oa = \frac{2l_1h}{3}(M + M_1) + hl_2\left(M + \frac{2M_2}{3}\right)$$

$$= h\left[\frac{2\times7.67}{3}(66.4 + 7.35) + 9.67\,(66.4 + \frac{2}{3}11.7)\right] w$$

$$= h(377 + 718)w = 1095hw$$

$$\therefore T = \frac{1095wh}{14.78h^2 + 25\left(\frac{1}{48} + \frac{E_c}{.244\,E_s}\right)} \quad . \quad . \quad . \quad . \quad (5)$$

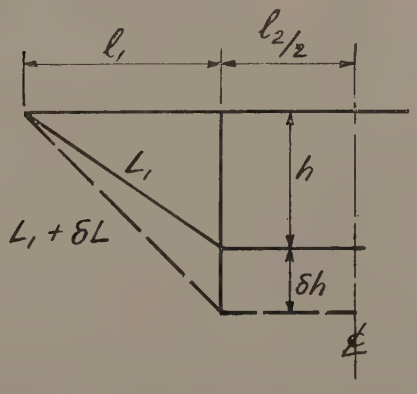
with $(E_c = 5\times10^6,\ E_s = 17\times10^6)$

$$T = \frac{1095wh}{14.78h^2 + 30.6}$$

Upward deflection of beam due to force in tie

$$y = \frac{M'l_1\left(l_2 + \frac{2l_1}{3}\right)}{2E_c\,I_c} \quad \text{where } M' = Th$$

$$y = \frac{M'\times92\left(116 + \frac{2}{3}\times92\right)}{2\times5\times10^6\times72} = \frac{2.27M^1}{10^5} \text{ inches} \quad . \quad (6)$$



$$\delta l = \frac{T(l_2 + 2L_1)}{2\,A_s\,E_s} = \frac{302T}{2\times.122\times17\times10^6} \qquad L_1^2 = 92^2 + 14^2$$

$$= \frac{.729T \text{ in.}}{10^4} \qquad L_1 = 93 \text{ in.}$$

$$h^2 = L_1^2 + l_1^2$$

$$(h + \delta h)^2 = (L_1 + \delta l)^2 - l_1^2$$

$$\therefore \delta h(2h + \delta h) = \delta l(2L_1 + \delta l)$$

$$\delta h = \frac{\delta l\,2L_1}{2h + \delta h} = \frac{0.729T\times2L_1}{10^4(2h + \delta h)}$$

$$= \frac{1.356T}{2h + \delta h} \times 10^{-2} \text{ ins.} \quad . \quad . \quad . \quad . \quad (7)$$

The trussed beam is prestressed by lengthening the struts. The relationship between the increase in strut length and the increase in the force is derived from equation (6) and (7)

If d = increase in strut length

T' = increase in tie force

Then $d = y + \delta h = \frac{2.27T^1h''}{10^5} + \frac{1.36T^1}{2h + \delta^L} \times 10^{-2''}$

Then $d = T'\left(\frac{2.27h}{10^5} + \frac{1.36}{200h}\right)''$ . . . . . (8)

neglecting such quantities of the 2nd order.

The tie forces and midspan moments for various stages of loading are derived from these equations and given below.

M_S = midspan moment $= M + M_2 - M'$

$= \frac{wl^2}{8} - Th$

sagging moment shewn + *ve*

Total dead load $w = 75$ lb./ft.
live load $w = 60$ lb./ft.

The dead load is applied in two stages of 35 lb./ft. and 40 lb./ft. respectively.

The adjustable struts are lengthened after the first part of the dead load, and again, when the total dead load, have been applied.

$w = 35$ lb./ft.
$T = 900$ lb. $M_S = +1700$ lb./ft.
(from equation 5)

Length of strut increased by $1\frac{3}{8}''$
$T' = 1650$ lb.
(equation 8)

$T = 900 + 1650 = 2550$ lb. $M_S = -500$ lb./ft.

$w = 75$ lb./ft.
$T = 1930 + 1650$ $M_S = +1300$ lb./ft.
$= 3580$ lb.

Length of strut increased by $1\frac{3}{8}''$
$T' = 1650$ lb. $M_S = -1500$ lb./ft.
$T = 3580 + 1650$
$= 5230$ lb.

$w = 75 + 60$
$= 135$ lb./ft.

$T = 3460 + 2 \times 1650$ $M_S = +1200$ lb./ft.
$= 6760$ lb.

Upward deflection of beam at midspan after second adjustment of the struts = $\frac{1}{2}''$.

Downward deflection of beam at midspan with full live load = $\frac{1}{4}''$.

Note.—For simplicity, the effect of creep in the concrete has been neglected in the above calculations. In actual fact the necessary adjustments to allow for a variation in the Modulus of Elasticity must be made.

Book Reviews

Prestressed Concrete Design and Construction, by F. Walley. (London : Ministry of Works, H.M.S.O., 1953.) $9\frac{3}{4}$ in. × $7\frac{1}{2}$ in. 279 pp. + xv. Price 30s.

The book commences with a brief review of early history and development and the methods by which prestressing is carried out.

The section on design, which deals mainly with beams, making only a brief reference to poles and columns is clearly set out without an undue complexity of symbols. Students of the subject and designers should benefit from a study of the principles and examples demonstrated.

Aspects of the ultimate theory are discussed under the heading " Prestressed concrete under load." The tables showing failing loads for actual tests on beams will be a useful guide in developing practical design.

The chapters on materials, allowable stresses and losses bring together in very readable form the essentials of the present state of knowledge on these subjects.

The best known English systems of prestressing are described and it is unfortunate that publication was presumably too early to allow mention of the Gifford-Udell jack. In future editions of the book the section on practice might well be extended to include notes on some of the troubles that occur on site and the principle of repairing damaged sections under artificial relief of stress.

The essentials of some special design problems are carefully explained including composite beams, two stage prestressing, circular tanks and pipes and continuity.

The appendices at the end will be considerable help in cutting down arithmetical work especially in so far as an approximate suitable section can be quickly picked out. A further appendix giving a selection of the major works carried out in this country would have been valuable to students.

The book is a useful contribution to the literature on prestressed concrete.

D. H. N.

Concrete Farm Structures, by A. M. Pennington. (London : Concrete Publications Ltd., 1954.) 153 + vi pp. 9 in. × 6 in. Price 12s.

An invaluable book to both the country practitioner and the stock-breeder—whether he be farmer, smallholder or just cottager. The descriptive text is amply illustrated with drawings and details which are well dimensioned and easy to follow. Written in ' easy to understand ' language it is not just ' another text-book ' but a valuable addition to the bookshelves of the countryman, and contains a wealth of information which one would not expect to find in a book bearing the prosaic title " Concrete Farm Structures."

Nevertheless, I would suggest that the user of the present edition would be well advised to consider in more detail the details of the roofs of stock-houses. Generally I consider these are too thin and are rather disproportionate to the warm snugness of the walls and floors. Condensation brought about by inadequate attention to roof coverings is the origin of much distress amongst animals.

R. J. V.

Tragfähigkeit Metallischer Baukorper in Bautechnik nd Maschinenbau, by Dr. Ing. Karl Helmut Ruhl. (Vilhelm Ernst and Son, Berlin.) Price 24 D.M.

This book is described in the sub-title as a survey of .e carrying capacity of steel and light alloys. It is ncerned with members of both structures and achines. After a brief treatment of the conception " safety," the possibility of structural damage, and .e load factor, the author goes on to examine the eans available for the investigation of states of stress. pparatus for the dynamic load, X-ray and photo-astic analysis, electric resistance strain gauges and eir use on test pieces and complete members come nder adequate, if brief, review. The second chapter nds with an explanation of the uses to which strain easurements can be put in the determination of states stress.

Chapter 3 shows how the results of the study of stress e modified by the shape of the member in question. rooved shafts, perforated plates, non-circular sections nder torsion are all studied, and tables and curves for ie design of some of these members are provided.

Chapter 4 is short, dealing with residual or " locked-p " stress, but it contains, as do all the other chapters, list of references which allow of further study of the pic. Chapter 6 follows later with a treatment of the rrying capacity of members sustaining residual resses and with the problem of embrittlement.

Chapter 5, although dealing only with static loading, ves a comprehensive review of theories of plastic ilure in an ideally plastic material. Bodies of various apes and subjected to various loadings are studied. valuable part of this chapter is the section sum-arising and evaluating the most important theories f failure and showing how they can be represented raphically.

In Chapter 7 the problem of load carrying under peated loading receives a generous treatment, and ie influences of the properties of the material and of ie shape of the member are shown to have effect on ie final result. The extent of their mutual dependence studied. Stresses acting along several axes and the uestion of residual stresses are again touched upon. he book ends with a chapter on Dynamic and transient ading and with a comprehensive summary of the work overed.

This is an unusual book, for although it treats of the trength of Materials it does not do so in the conven-ional manner. Modern in outlook, the author has not nerely presented mathematical theory, but has also ealt with the practical measurement of strain and with he meaning of the results so obtained. As must be in ny book professing to study total carrying capacity of netallic members, the chapters on plastic failure and n repeated and dynamic loading are of considerable nportance. The book is written in German and lthough a great deal of the matter is mathematical and hus comprehensible by most engineers, some know-edge of the language is necessary if the full value of he book is to be extracted.

W. F. C.

Comparative Bridge Designs, edited by J. G. Clark. Cleveland, Ohio : The James F. Lincoln Arc Welding oundation.) 211 pp., 8½ in. × 6 in. Price $2.50.

This book is the third in a series presenting the esults of Award Programmes, organised by the James . Lincoln Arc Welding Foundation, to encourage nterest and development in the design of welded ridges.

The present volume contains a selection of the material contributed in the 1952 Programme which invited comparative designs between riveted and welded highway bridges. The aim of this Programme was to demonstrate the economies that could be achieved, in both steel and overall cost, by the use of arc welding in place of riveting.

Apart from ensuing that the riveted and welded designs were based on similar loading and construction conditions the organisers imposed no limit on span or type of bridge, and consequently the book includes examples of single and multiple span bridges of simple and continuous girder or truss construction. The spans vary between 32 ft. and 1,800 ft. total.

There are many ideas described and illustrated which could be of value to those interested in welded bridge design.

H. A. C.

Text-book of the Materials of Engineering, 8th Edition, by H. F. Moore and M. B. Moore. (New York & London : McGraw-Hill, 1953.) 372 + xi pp. 9 in. × 6 in. Price 48s.

This book, first published in 1917, has been brought up-to-date in the present 8th edition. The first part of the book deals concisely with strength of materials, failure by flow or creep, by fracture and by corrosion and wear. Then the general properties and manufacture of the common engineering materials are considered, followed by chapters on testing and inspection and specifications, mainly, of course, in American practice. The book concludes with a chapter on stress, strain, atomic and crystalline distortion and structural damage to machine parts and structural members, which gives a brief general discussion of the present position. References for additional study, mainly American, are given at the end of each chapter, and also a set of questions which forms a good test of the reader's appreciation of the subject matter of the chapter.

Whilst intended primarily for use in technical colleges, the book is also of considerable interest to practising engineers.

Building Construction and Drawing. Part I, Elementary Course, 21st Edition, by G. A. and A. M. Mitchell. (London : Batsford, 1953.) 648 + viii pp., 7½ in. × 5 in. Price 15s.

' Mitchell ' needs no introduction to the established architect or builder for it is found upon all their bookshelves. Although it is regarded, and intended, as primarily a ' student's ' book in the sense that it is an invaluable guide to the tyro and is usually bought as such, it remains an invaluable guide to the practitioner. In it are all the niggly details so easily forgotten and yet so often required in both design and practice.

The edition under review has been brought thoroughly up-to-date and is of the same high standard as its predecessors. Modern development and methods are fully represented within the scope of its title and within its pages is a valuable foundation for later and more detailed study. To the practitioner I would suggest that he looks at his existing copy which, like mine, has probably been on his bookshelves for some years, and, by comparing it with this edition, he will find demonstrated in a simple but telling way how things have advanced since his student days. In other words, many ' Mitchells ' still in use should be replaced by a copy of this edition with profit and interest.

R. J. V.

(Continued on page 66)

Various Types of Wharves and Marine Structures Recently Carried out in France*

By L. P. Brice, M.Soc.C.E. (France)

Introduction

For many years, jetties and other light piers were, by tradition, built in timber. This material, however, presents difficulties in maintenance, is subject to rot and prone to attack from the teredo which all contribute in shortening the life of timber structures. Consequently it seemed reasonable to replace timber by ferro-concrete.

This change over had been thought of and partly carried out in the early stages of ferro-concrete design. About 1910 Mr. Quost had built at Ivry, near Paris, some piers guiding the entrance to a lock, which are still in service. In London, L. G. Mouchel had by that time been responsible for the designs of numerous ferro-concrete quays and piers, such as, for instance, the submarine base at Haslar, built in 1907 at the entrance to Portsmouth harbour.

This achievement, outstanding for its time, consisted of an 850 ft. long wharf with a 450 ft. long jetty and other ancillary works. The deck and the beams were cast in situ and supported on precast ferro-concrete piles more than 40 ft. long.

It is typical of any new form of material that it begins by adapting itself to the shapes and uses of the one it replaces. Ferro-concrete was no exception, and it began as some new mode of wood. The structures we have referred to were merely timber constructions conceived in timber and carried out in ferro-concrete.

This led, as a result, to some misunderstandings : the new material having come of age should be used by taking fully into account its own salient features, qualities and deficiencies. It is therefore advisable to survey these characteristics in the light of their application to marine structures :

1° Ferro-concrete is a material which needs special care against corrosion, particularly on the sharp edges, the projecting parts and any recesses where water can remain stagnant.

2° Sea-setting concrete is unreliable due to risks of washing away, and should consequently be poured in water-tight shutterings.

3° Ferro-concrete being a material of low flexibility, any impacts may produce cracks leading eventually to additional corrosion.

Fig. 1.—Wharf at Sète

A few simple rules may be evolved from these observations and we shall endeavour to bring them to light by comparing two relatively old structures with some new ones we have recently carried out.

The two old structures, which will be described rapidly, are the wharf of the Compagnie Bordelaise de Produits Chimiques, built at Sète (Herault) between 1928 and 1930, and the eastern jetty at Dunkirk which we built in 1932.

The recent marine works we shall describe are six wharves or jetties built between 1948 and 1953 in the harbours of Brest, of Etel, of La Trinite-sur-Mer and of Dunkirk.

**Paper read before the Institution of Structural Engineers and the British Section of the Société des Ingenieurs Civils de France at 11, Upper Belgrave Street, London, S.W.1 on Thursday, 12th November, 1953. Lt.-Colonel R. F. Galbraith, M.C., B.Sc., M.I.Struct.E., A.M.I.C.E. in the Chair.*

The Wharf at Sète

We were commissioned to build, in a direction slightly normal to the shore, a wharf some 410 ft. long and 52 ft. wide to serve the loading and unloading of cargo boats.

At some 26 ft. below sea level, the sea bed consisted of about 4 ft. of mud, then a layer of sand mixed with sea shells down to a level of 46 ft. below sea level where sound grey clay was found.

It was decided to design the foundations on this sound grey clay, as the bearing capacity of the ground above was not considered sufficient.

When completed, the deck was to be 7 ft. above water level, which, at Sète, is practically constant. This restricted headroom precluded the adoption of the classical shuttering methods. We submitted a scheme which proved acceptable to our clients and which may be described as follows :

The wharf is supported by 22 pillars which, together with the trimmer beams, form 11 piers some 36 ft. apart. The deck with its ribs and its 5 in. slab rests on these trimmer beams. The distinctive originality of the design lay in the fact that the work was almost entirely prefabricated.

The pillars consisted of 50 ft. long ferro-concrete cylindrical shells 7 ft. in diameter with a 12 ft. diameter base provided with a sharp edge to facilitate penetration into the ground. This cutting edge consisted of a $\frac{3}{16}$th in. thick steel shoe riveted to the base.

We availed ourselves of the constant water level to design the pillars as prefabricated elements made and sunk above their final position. Cast iron blocks embedded in the cylinder walls enabled the hoisting and setting out of the pillars in their correct position.

The sinking of the pillars was assisted by the sharp edge of the base cutting through the sand until it remained anchored at the correct level in the clay. The cylinders were then filled with concrete.

Upon these pillars rested the deck which was in turn partly prefabricated.

On the pillars, we laid the lower part of the beams, which was prefabricated to a depth of 30 ins. It was then strong enough to support the prefabricated ribs. The upper part of the trimmer beams and the deck slab, cast in situ, made the ensemble monolithic and complemented the resistance of its various components.

The connections were easy to cast, stub bars having been left in the prefabricated elements. The weights of the various components were :

Cylindrical shells ..	32 tons
Prefabricated beams	13 tons
Prefabricated ribs ..	7 tons

The wharf has given satisfactory and uninterrupted service for the last twenty-three years.

The Eastern Jetty at Dunkirk

We shall only mention a few relevant facts concerning this jetty which is partly outside the scope of the present subject.

It had been decided to lengthen the existing jetty, continuous on massive caisson foundations with a timber superstructure. The project which was submitted to us merely specified a cast in situ concrete superstructure instead of the timber one.

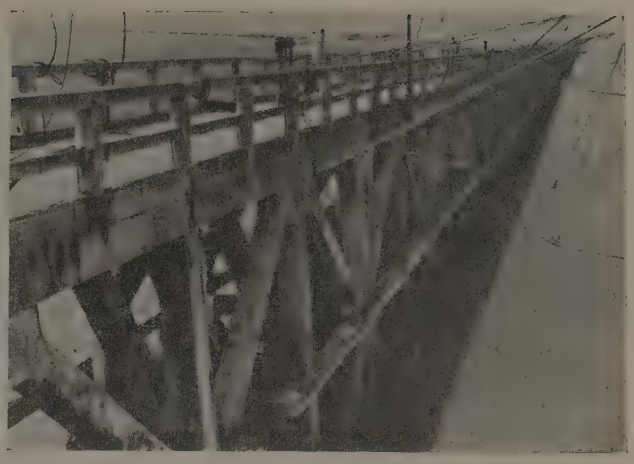


Fig. 2.—Eastern Jetty, Dunkirk

Our proposal offered to build the jetty in two parts : prefabricated transversal frames with cast in situ longitudinal bracings. The deck was also prefabricated.

Our scheme was finally accepted and we carried it out during the years 1931 and 1932. It has given complete satisfaction.

The main part of the jetty stands today in a very good condition. Some damage remains, however, from German bombings during the evacuation in 1940.

It may be of interest to note the very marked difference between the appearance of the precast frames and the cast in situ bracings, the compactness of the precast concrete being always superior to the one cast in situ.

The Landing Stage of the East Mole at Brest

The landing stage, built before the war above the slope of the east mole in the docks, had been totally destroyed by the Germans in August 1944.

To lengthen it at reduced costs, triangular frames, normal to the landing face, had been assembled and braced together. They supported the deck and the two rail beams of the crane.

The resumption of harbour traffic in 1949 demanded its reconstruction. In order to avoid clearing the existing debris, the setting-out of the piles had to be designed with a certain amount of flexibility and tenders were invited for a project which answered this condition. It included a mushroom floor, and a face wall preventing small craft from entering under the landing stage.

In view of the shuttering and scaffolding difficulties involved, we endeavoured to avoid the suggested mushrooms below the deck and we obtained finally the adoption of the following method of working :

1° Pile driving carried out, partly from the sea side with a floating driver, and partly from the retaining wall of the mole for the raking piles.

2° Provision of a flat horizontal shuttering held by collars placed at the top of the piles.

3° Lopping the piles to the level of the deck.

4° Casting the mushrooms above deck within the depth of the filling specified, using a very simple prefabricated reinforcement.

Fig. 3.—Landing Stage of the East Mole at Brest

During the first stage of operations, the concrete was pumped into the deck slab, and, during the second stage, a special repetition form of shuttering was used to cast the mushrooms.

At the present time, only the first section of the landing stage is completed : it is 538 ft. long and 87 ft. wide. It was built in twelve months during the year 1950.

The Jetties of Etel and of La Trinite-sur-Mer

Etel and La Trinite-sur-Mer are two fishing harbours in the Morbihan, in south-west Britanny. The new jetties were built to improve working conditions. The first one was built at Etel.

The fishing harbour at Etel consisted of an unsheltered tidal dock and a 318 ft. long mole sheltering an area of water of less than an acre. Besides, the landing strips were insufficient.

In principle, the new scheme entailed :

1° Filling up the existing dock and retaining it by means of a new quay and slipway.

2° Sheltering the new harbour with a new breakwater jetty.

Tenders were invited for a jetty consisting of a ribbed deck supported on five rows of 16 in. piles.

We preferred to adopt greater spans with four rows of 22 in. octagonal piles, and a deck partly prefabricated. The work was carried out in the following stages :

1° Driving and lopping the piles.

2° Transversal beams cast in situ with upper stub bars.

3° Prefabricated face beams placed in position and connected to the transversal beams.

4° Prefabricated deck slabs placed in position and connected together.

5° Copings and other finishes.

The stub reinforcement was designed to avoid bending on site and only required the placing of the prefabricated pins and stirrups.

We intended at first to precast the transversal beams, but possible displacements of the piles, driven through a fresh stone bedding, led us to cast them in situ.

Driving operations were started in July 1951, and the placing in position of the precast elements continued in May 1952, the jetty being used in parts until its completion in September 1952.

The success achieved at Etel prompted us to carry out the jetty at La Trinite-sur-Mer on similar lines.

The jetty at La Trinite-sur-Mer is 192 ft. long and 33 ft. wide. It differs mainly from the Etel jetty in the provision of longitudinal spandril walls.

Driving operations were started in March 1952, and the work was completed in July 1952 when the fenders were placed in position.

The Dunkirk Oil Dock

Extensions in the refinery of the Societe Generale des Huiles de Petrole at Dunkirk brought in more and more tankers, and the old berths, located in Dock No. 4, in the middle of the harbour, had to be abandoned.

In agreement with the Harbour Authorities, the Societe Generale des Huiles de Petrole (B.P.) decided to turn into an oil dock the evolution dock which was opened before the war when the new marine lock and the new outer harbour were built.

The complete scheme, as carried out, entailed :

1° A twin berth landing stage for the tankers, called " Tankers' Wharf," complete with access bridge, for the delivery of crude oil from the heavy 35,000 ton ships.

2° A " Coasters' Wharf " for future reshipments.

3° Ancillary works for the intake and delivery of 1,500,000 gallons per hour of cooling water, and the discharge of the same quantity of hot water.

4° A heavy dolphin to prevent the tankers from approaching the ruins of the destroyed submarine base. This structure also supports one end of the floating boom designed to isolate the oil dock in case of fire.

Fig. 4.—Tankers' Wharf—Handling of a precast pipe for cooling water system

Fig. 5.—Tankers' Wharf—Handling of a precast beam for the access bridge

We shall only describe the two landing stages, beginning with the " Tankers' Wharf " which was built first, during the period from January 1949 to March 1951.

The " Tankers' Wharf " consists of the three following distinct structures :

1° The access bridge abutment, anchored to the shore between two rows of sheet piles, and housing the suction pump of the cold water syphon.

2° A 460 ft. long access bridge 33 ft. wide.

3° A 312 ft. long landing stage 131 ft. wide for the mooring of large tankers, and fitted with 12 bollards, 8 shock absorbing fenders, and suitable gantries for the hoisting and handling of the delivery hoses.

Deck level of the access bridge and of the landing stage is at +26 ft. with anchorage at —26 ft., the water level in the dock fluctuating between +13 ft. and +20 ft.

The structures resting on piled foundations, it was intended to absorb the horizontal thrusts by raking piles, but their length precluded the adoption of this method. The ground was of poor quality, and hard clay with a sound bearing capacity could only be found at level —82 ft. We had to drive some 105 ft. long piles, and raking piles of this length would have been very difficult to drive. Consequently, we decided to absorb the horizontal thrusts by means of two gabions consisting of two heavy sheet-piled cofferdams filled with sand.

Fig. 6.—A view of Tankers' Wharf, commissioned

Some 82 ft. long sheet piles were driven to almost clay level, at —62 ft., on a layer of sand and sea-shells. They were then lopped to the correct level. A floating driver was used for the vertical piles, and a derrick with a fixed scaffolding for the sheet piles. The heavily reinforced deck slab, designed as a mushroom floor, rested on the 105 ft. long piles.

The access bridge rested on 8 piers and a fixed abutment. It consisted of a central 10 ft. wide roadway, two strips of piping for the delivery of the crude and refined products, and two suction pipes for the supply of cooling water to the refinery. Prefabricated elements were used as much as possible.

Each pier was concreted in parts, beginning with the prefabricate pipe supports placed in position, then the precast pipes and finally the longitudinal main beams which also were precast.

The upper part of the piers, the cantilevers and the deck slab were cast in situ.

A 60 ton derrick was used for hoisting and handling the 40 in. diameter precast concrete pipes. Each length of pipe was 62 ft. long and weighed 26 tons.

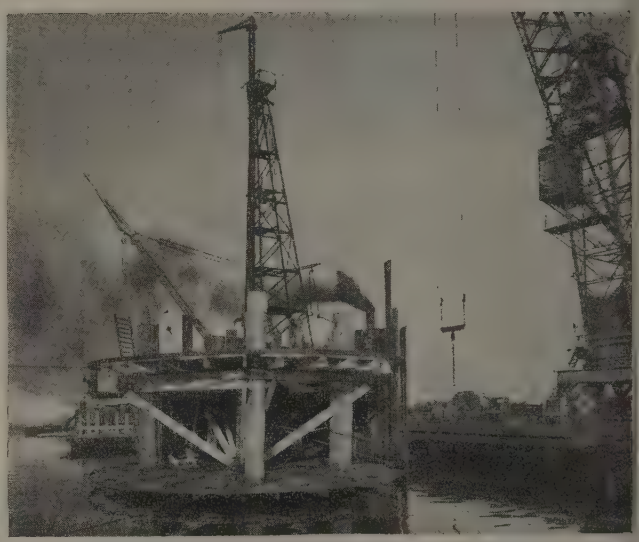


Fig. 7.—Coasters' Wharf—Building of a sheet pile gabion

Fig. 8.—Coasters' Wharf—Placing of a beam

Fig. 9.—Fitting-out quay at Dunkirk—Construction of a caisson in the floating dock

Fig. 10.— Fitting-out quay at Dunkirk — Towing of a caisson into position

The main beams were hoisted and handled in the same manner. Each precast concrete beam was 62 ft. long and weighed 25 tons. Openings in the web were provided at each end of the beams to make way for the reinforcement of the connections.

All these various operations were carried out satisfactorily without unforeseen difficulties, and the work was completed on time during the summer of 1951 before the opening of the new refinery.

We shall now describe the " Coasters' Wharf " built during the period from December 1951 to July 1952. It consists of a 682 ft. long deck 36 ft. wide supported on 12 piers and an abutment.

Each second pier consists of a 30 ft. diameter gabion with Senelle type of sheet piling, and carries a 50 ton bollard.

Concrete bodies resting on piles form the intermediate piers.

The deck consists of two longitudinal main beams which are prefabricated in the form of a vertical web and a horizontal walkway, and of a deck slab cast in situ between them.

The work was carried out in the following stages :

1° Construction of the piers.

2° Laying the prefabricated main beams.

3° Placing in position the prefabricated lateral face walls.

4° Casting the upper part of the piers, the ribs and the slabs.

Execution was facilitated by the experience gained during the construction of the " Tankers' Wharf " and by the loan of a 100 ton crane belonging to the Chamber of Commerce.

Fitting out Quay at Dunkirk

The Ateliers et Chantiers de France commissioned us to build this quay for the completion and fitting out of their ships. It is located in their shipyards at the rear of the old outer harbour, and, for easy service, as an extension to the existing crane track.

The hulls can berth there directly after launching from the adjoining slipways.

The quay will eventually be 1,312 ft. long, so that two large vessels can be completed and fitted out simultaneously.

The old masonry wall, which had been systematically destroyed by explosive charges placed in the rear, was already inadequate, its base being at level —15 ft. in a dock where the water level varies between +23 ft. and +16 ins.

The existing debris would have demanded expensive excavation and dredging work if the quay were to be rebuilt on its former site.

Instead, the new structure was built in front of the old one, some 60 to 80 ft. away. It consisted of a crane track designed for 20 ton cranes at 80 ft. spans.

We designed the first section, some 526 ft. long, in 4 spans of 115 ft. with two 33 ft. cantilevers, thus requiring 5 piers in all.

The deck consists of a central roadway between the two rail beams.

The piers, 115 ft. apart, are supported on 40 ft. diameter gabions consisting of sheet-piled cofferdams filled with sand, and with a ferro-concrete coping.

The reinforcement of the coping was prefabricated and hoisted in position by a floating crane.

The rail beams were large, and weighed 150 tons, so we designed them as caissons which were prefabricated in a floating dock, then launched and towed to their correct position.

These 90 ft. long caissons are 25 ft. wide and weigh 700 tons excluding the weight of the deck. They were made on the opposite side of the dock, released at high tide from the floating dock and towed to their supports, their buoyancy being increased by temporary lateral steel floats.

The whole operation lasted only an hour, between the time the caisson was released until it was firmly placed in its correct position on the gabions.

(Translated by Mr. P. J. Gerard, M.I.Struct.E., M.Soc.C.E. (France)

The following additional information has been supplied by the author on request :

(1) There are no special regulations for marine structures in France, beyond the general regulations for reinforced concrete.

(2) The concrete mix was usually 25 lbs. per cu. ft. of concrete, approximating to 1:1½:3 mix.

The stress in the concrete was 925 lbs. per sq. in.

The stress in the steel was 18,500 lbs. per sq. in.

(3) The cover of concrete now used as compared with the cover used in earlier structures is as under :

(a) under sea level, previously 30 mm. now 50 mm. ;

(b) above sea level, previously 20 mm. now 30 mm.

(4) The mooring strains were from 17 to 165 metric tons. Impact was from 55 to 275 metric tons.

Drawings and calculations for the caisson used in the Dunkirk Quay have been placed in the Library of the Institution of Structural Engineers for reference.

Discussion

A joint meeting of the Institution of Structural Engineers and the British Section of the Société des Ingénieurs Civils de France was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 12th November, 1953. Lt.-Colonel R. F. Galbraith, M.C., B.Sc., A.M.I.C.E., President of the Institution, was in the Chair.

A paper was presented by Monsieur L. P. Brice, M.Soc.C.E.(France) entitled " Various Types of Wharves and Marine Structures recently carried out in France " ; and those structures were illustrated by many drawings and photographs.

The President, opening the meeting, first welcomed all members of the British Section of the Société and said how pleasing it was to see how many were present. He felt sure they would all enjoy the paper ; and M. Brice responded.

The President, formally introducing the author, said he was educated at the Ecole Centrale des Arts et Manufactures, Paris, the foremost school for engineers in France ; and when he left the school he had entered his father's firm of Sainrapt and Brice.

During the years 1932-34 he was able to spend part of his time as a Professor at the Ecole Centrale des Arts et Manufactures dealing with the design of ferro-concrete work, and a large proportion of his activities had been devoted to research in the field of resistance of materials.

In 1924 he took part in a series of tests carried out during the preparation of the new French regulations for ferro-concrete work, and conducted some remarkable experimental research work to determine the behaviour of internal stresses in concrete.

In his role of Consulting Engineer in 1936 he presented to the Second International Congress on Bridge and Structural Engineering, in Berlin, a report on the function of plasticity in the stability of constructional work.

Those members of the Institution and the Société who had attended the Congress of the International Association for Bridge and Structural Engineering held at Cambridge in 1952 were conversant with M. Brice's preliminary report on the relation between the formation of cracks in ferro-concrete beams subjected to bending, the modular ratio and the value of residual strain.

At the present time M. Brice was assisting in the drafting of some new regulations on ferro-concrete design in France ; and he had brought to the Institution of Structural Engineers, on behalf of his Association, an invitation to nominate a delegate to attend the conference towards the end of November. The President hoped the Institution would be able to send delegates.

Mr. Peter Gerard, who had translated the paper, presented it on behalf of the author.

(The paper is printed on page 56.)

Discussion

THE PRESIDENT, proposing a very hearty vote of thanks to M. Brice for his paper and to Mr. Peter Gerard for having translated and presented it, said it was most interesting, dealing as it did with various maritime structures constructed in France, many of them since the war.

The photographs of the eastern jetty at Dunkirk brought back to the President's memory the last two occasions on which he had seen it. The first occasion was on 1st June, 1940, when it was under shell fire by the Germans, and one was very glad to board a destroyer and to leave it behind. The second occasion was about 5th May, 1945, when Dunkirk had been relieved, and he was able to see how heavily the jetty had been mined by the Germans. The Allies had then a company of German sappers at their disposal and had given them the task of clearing their own mines, much to their annoyance.

Expressing his interest in the extent to which prefabrication was adopted by French engineers for maritime structures, as shown by the paper, the President said that in England we rather tended to fight shy of prefabricated structures in some cases or, if we did prefabricate, we were rather limited by the plant and equipment available. M. Brice was happily prefabricating beams weighing 30 tons, and the last caisson weighed 700 tons. That must be well on the way to a record.

The President formally moved a most hearty vote of thanks.

Mr. J. E. SWINDLEHURST, O.B.E. (Past President, Institution of Structural Engineers) seconded with much pleasure the vote of thanks to M. Brice for his most illuminating paper, and to Mr. Peter Gerard for the very eloquent and interesting manner in which he had presented it.

M. Brice, he continued, was quite an old friend. As long ago as 1st February, 1938, he had presented a paper to a joint meeting of the Institution of Civil Engineers, the Institution of Structural Engineers and the Société on " Dunkirk Harbour Extension Works." Then he had given the further benefit of his experience on 21st April, 1949, when again there was a joint meeting of the Société and the Institution of Structural Engineers, the subject on that occasion being " The Reconstruction of the P. H. Watier Lock at Dunkirk." The author of the paper was M. Le Gorgeu, the Engineer of the works. On that occasion a colleague of M. Brice, namely, M. de Jarny, had read the paper in English.

Again, on 13th December, 1951, at another joint meeting of the Société and the Institution, a paper by M. Brice on " The Construction of the Temple Barrage on the River Lot " was presented. It was unfortunate that on that day weather conditions had prevented his attendance ; but again M. de Jarny had stepped into the breach and all was well.

Mr. Swindlehurst expressed sincere gratitude on behalf of the meeting for the present paper, presented for their education and instruction. There were many points in it, he said, which merited still further elucidation, and no doubt they would be brought out in the course of discussion. As the President had mentioned, prefabrication had been carried to a remarkable degree in the structures that were illustrated. The meeting was most indebted to M. Brice for having come to this country and for having prepared the paper.

(The vote of thanks was accorded with acclamation.)

Mr. B. A. E. HILEY asked the President's permission to show three slides. He then said that M. Brice had mentioned Messrs. L. G. Mouchel & Partners' work at the Submarine Base at Haslar at the entrance to Portsmouth Harbour, carried out in the year 1907, and said that he thought that an earlier work designed on the London River might be of special interest, since this work was designed in 1900 and constructed in 1901/2. The works were of similar general design to those presented by M. Brice in his paper.

He referred to the Jetty at Dagenham Docks, Essex, for the owners Messrs. Williams & Sons Ltd., whose Mr. Kirwan was at the meeting being discussed.

Slide 1 showed the work at Dagenham now over half a century old. This coaling jetty was originally 458 ft. long with an approach of some 310 ft. The average height from river bed to decking now averages some 60 ft. The jetty was designed to accommodate four Transporter Cranes each of 40 tons gross weight and 3 tons capacity. The picture was taken during construction. The approach and jetty itself carried standard gauge rail tracks.

Slide 2 showed this same structure as seen in October 1953. One could see that the 4/40 ton cranes had disappeared and were replaced by 2/160 tons gross weight and 7½ tons capacity, long jib grabbing cranes. Not only had this structure been actively working as a coaling jetty during its existence but latterly had been called upon to do heavier work than that for which it was designed.

Slide 3 In order to stiffen the deck supporting these 160 ton cranes the client had used Bailey Bridge units spanning between the supporting columns. In order to give bearing support to these units it was necessary to drill the concrete through 6 ft. diameter column heads and the efforts made could be seen in this slide. Concrete of such density and strength was used in 1901/2 that every inch had to be drilled and in the process drills broke and required much resharpening. In fact, little progress was made until special Tungsten-carbide steel bits were obtained.

Other factors in this construction that were of interest were :

1. The piles were cast vertically.
2. The principle of encasing groups of piles in cylinders was first established in this country.

Today, of course, piles were cast horizontally on a concrete bed and the results were quite as good. Nevertheless it was interesting to note what was done half a century ago.

Mr. Hiley went on to say that on looking over his records—they had what they called their Blue Book illustrating all these early works from 1898 to 1921, all properly photographed and described—in the River Thames alone he found several references to structures now quite old, the dimensions of which varied from 1,000 ft. long to 2,563 ft., some with adjoining transit sheds of similar length and some double deckers, and all accommodating ships of considerable size. All these works were still doing duty and compared with such structures as had been illustrated by M. Brice's paper.

The difference, however, between the structures illustrated by M. Brice and those Mr. Hiley had mentioned was generally that in M. Brice's works he had developed precast units to a remarkable extent. The points in common, however, were that both had precast piles and both used cylinders to encase piling groups.

Mr. Hiley continued to say that whilst he appreciated the saving in time by using so many precast units, were we not deviating rather drastically from the earlier conception of a monolithic structure? He had noticed that connections were looked after in M. Brice's structures by the steel arrangement, but in the in situ concreting was there not some danger in making good connections?

In again paying tribute to M. Brice, whose paper was one of great interest, Mr. Hiley said he would like to refer to Mr. Tony Gueritte. Someone had once said to Mr. Hiley when talking of his old friend: "He is one of those good engineers who goes to France and lectures on the wonderful work carried out in Britain and then returns to Britain and tells us of the wonderful works carried out by French engineers in France, and by this exchange and kindly references to the engineers of both countries, he stimulates British and French engineers to go one better." Such a man as Mr. Gueritte, the gentleman who founded the British Section of the Société des Ingénieurs Civils de France, had done more than anyone Mr. Hiley could recall, to cement the relationship between our two countries, not only in engineering circles, but socially as well.

Mr. J. PALMER said how very glad he was to join in welcoming his old friends M. Brice and M. de Jarny in London again.

It had been a great pleasure, he said, to have Messrs. Sainrapt & Brice as contractors on a very difficult job in another part of the world. But he was sorry to say that the job had developed considerable troubles, due largely to the bankruptcy of the client, not the bankruptcy of the contractors nor of the consulting engineers.

The "gabions," consisting of two heavy sheet-piled cofferdams filled with sand, to absorb horizontal thrust at the "Tankers' Wharf" at the Dunkirk Oil Dock, were extremely interesting. He did not know the English word for "gabion," but it might be "strong-point." However, from the illustrations evidently they were built to form a number of strong points to resist the horizontal thrust imposed by tankers. He asked what was the amount of the horizontal thrust those cylinders filled with sand were designed to take.

Mr. PETER GERARD ascertained from M. Brice that the thrust was of the order of 100 tons.

Written answer.:

In the structures described the gabions were of different sizes. The thrust at the bottom of the gabion varied between 2.75 and 6.9 tons per sq. ft. The traction in the specially designed flat sheet piles varied between 34 and 131 tons per lin. yard of joint (the resistance of the joint being 275 tons per lin. yard).

As far as the French word "gabion" is concerned it refers more to the way the strong point is built than to its function. The origin of the French word is found in the Italian word "Gabbione," which refers to a kind of basket without bottom used as a protection by the soldiers when placed side by side and filled with earth.

Mr. D. H. LITTLE asked why prefabrication was adopted, whether for the purpose of saving money or time. He had been impressed by the rapidity with which the structures described were built. For example, one was started in March and was finished in June. He asked what was meant by the words "starting" and "finishing." Did they mean that the work was put out to tender and the contractors started work in March, or had there been a great deal of preparatory work beforehand, so that the period from March to June represented the actual time on the site? There might have been a good deal of work done before March in making the prefabricated units.

Mr. GERARD said prefabrication was adopted in order to achieve a better quality of work. The elements were prefabricated before March, but only 1½ months before, to give time for the concrete to be of absolutely first-class quality when the elements were placed in position. The essential reason why prefabrication was adopted to such an extent was demonstrated by Slide 17, showing one of the structures at Dunkirk. In the prefabricated elements the concrete was much better than that in the bracings at the top which were cast in situ. By prefabricating the elements one could ensure that the conditions in which they were made were such as to give the best quality of concrete.

Mr. C. GUILLEMIN TARAYRE, dealing with cost, said that the total cost of hiring the floating dock and towing in position of the 700 ton caissons when divided by the number of cubic yards of concrete in the caissons amounted to £3 per cu. yard. That was to be compared with the cost per cu. yard of concrete of staging on timber piles and scaffolding and also supplementing the shuttering to ensure watertightness, for in normal parctice a very large part of the concrete would have been poured at below sea water level. So that the cost when using prefabricated elements was considered to be very cheap.

Mr. J. BAK, discussing the horizontal thrust imposed by tankers against the quay at Dunkirk, which was said to be about 100 tons, asked what form of shock absorbers were used to absorb the energy of impact. Also, he asked whether the author had used rubber fender tubes or buffers of the type used in the United States and applied recently for berthing large ships in the New York Harbour.

M. C. Guillemin Tarayre said that he thought springs had been used, but he could not say much about the device finally adopted which was designed by SGHP.

Mr. B. F. SAURIN gave some information about the shock absorbers used on the jetty. The variation of the water level in the dock, he said, was not very great, so that sufficient shock absorbing capacity could be provided at the level of the jetty deck to accommodate the freeboard of all vessels likely to arrive there.

The jetty had a fairly thick deck, and at the point of impact a shock absorbing panel about 4 ft. wide by 5 ft. high was provided. That panel consisted of an assembly of rubber materials in three layers.

The first layer, i.e., that which came against the ship, consisted of four lengths of thick rubber tubing mounted horizontally one above the other, each having a diameter of approximately 15 in. and a central hole of about 4 in. diameter. Behind the four tubes there were four vertical 12 in. square-section rubber elements, also having a central hole. The same material was used in the third layer, but in the other direction. The whole assembly was held in a steel frame suspended by chains in a chase in the concrete.

He understood that the total energy absorbing capacity of the assembly was 27 ton-metres, i.e. approximately 1,000 inch-tons.

Professor A. L. L. Baker asked what crushing strength in the concrete was assumed in the design calculations, because it seemed to him that it was important to ensure the high quality of the concrete in order to reduce the weight of the prefabricated elements. Also what value of the factor of safety was used?

M. Brice (interpreted by Mr. Gerard) said it was difficult to talk of resistance unless a great number of tests were made; at Dunkirk, 300 or 400 tests were made, and curves were plotted for the concrete at the ages of 8, 28 and 90 days. The curves were almost parallel; whether a concrete was good or bad, its resistance would increase with age. The quality of the concrete was defined not only by the mean resistance, but also by the difference between the maximum and minimum. The ideal was to have a concrete in respect of which there was only a very small difference between maximum and minimum. He did not remember the figures exactly, but the minimum difference was about 5 per cent.

Mr. H. Kaylor asked for a little further detail concerning the placing of the caissons into position on the gabions, or strong points, which appeared in the illustration to be perhaps 10 or 12 ft. above water level at the moment the picture was taken.

Secondly, one of the illustrations showed an *in situ* portion of the work, where inverted caps or heads were used for a mushroom construction. He asked what was the particular reason for using the inverted type. Was it in order to provide clearance underneath, because the space above was eventually filled in with earth? The unique method of flat slab construction was interesting.

M. C. Guillemin Tarayre replied that in the part of the harbour in question the height of the tide varied up to a little over 20 ft., and the difference in level was used to place the caissons, choosing favourable times.

In reply to the second question, he said the arrangement used was arrived at in order to simplify the shuttering. It was the underside of the slab, of course, which had to be shuttered, and the more simple the shuttering, the more economical it was. On the other hand, by the use of clamps it was easy to support the flat underface of the slab. The mushroom device was designed to overcome the difficulty of driving the piles into accurate position due to the wreckage in the ground.

Mr. R. P. Mears said that in his experience factory-made precast concrete products were generally greatly superior to anything made in situ, and he believed that precast products could be used at very much higher stresses. He asked if it were also the experience of the French engineers that the prefabricated concrete gave considerably higher resistance figures than the in situ concrete, and whether that was borne out in the test results on cubes.

Watertight shuttering was not particularly easy to make. But in his recent experience of work below high tide level, vibration had been of enormous advantage in making the shuttering watertight, because the effect of plenty of vibration of well-placed concrete closed any interstices there might be in well-made shuttering, and the concrete was perfectly good when stripped.

M. C. Guillemin Tarayre considered it absolutely impossible to compare the resistance to crushing of concrete prefabricated in a factory and concrete made at site, the case of the concreting of the caissons (4) being more similar to the conditions of an ordinary site than to the case of a factory. The quality of concrete made on a works site should be compared with that made on another works site.

M. Brice's idea was to avoid the use of watertight shuttering by using sections of concrete precast; much better quality was obtained by precasting, and it enabled engineers to avoid the sort of defects that had occurred on the Dunkirk jetty.

Mr. C. H. Hockley, in view of the amount of precast concrete used in the works described in the paper, asked whether prestressing had been considered. One heard a good deal about the use of prestressed concrete in France, and he had wondered whether, in the works described in the paper, it had been ruled out on grounds of economy, or whether it was considered unsuitable as a form of construction for the jetties.

Mr. Peter Gerard said M. Brice had not used prestress in any of the works dealt with in the paper, but he had used it in some other works. It was simply a question of the balance sheet. Prestress was used in cases where it was desired to lighten the structures; but in marine works such as he had described one did not wish to lighten them, but on the contrary they should be as heavy as possible, within the limitations of the tender. For example, when a ferry had crashed into one of the dolphins in Dunkirk harbour in a gale, the passengers had had to wait on board for twenty-four hours before they were able to land. So that prestress did not seem to find so much application in marine work as it did inland.

Mr. J. N. McFeeters, asking for information concerning the method of tendering in France, said he had gathered that the controlling authority issued designs to contractors, who could quote lump sum prices or unit prices in competition.

With regard to defects which might arise after completion of the works, he believed that in France there was a legal requirement in respect of shore works that over a period of 10 years after completion the contractor had to repair defects. He asked if that operated in respect of the works described in the paper.

Finally, Mr. McFeeters expressed his great interest in the illustrations of the works at Dunkirk Harbour. The tanker jetty there was completed for an associate Company of the Anglo-Iranian Oil Company. He had seen that jetty during construction and since, and he considered it to be elegant and to the credit of all concerned.

M. C. Guillemin-Tarayre, replying, said that tenders were invited for all the works, except the last, described in the paper, and the contracts were given on the basis of unit prices. In the last case, there was just a private agreement, and tenders were not invited.

On the question of design, most of the works were designed on the basis of the plans of M. Brice.

After completion of the works, the builders had responsibility for 10 years.

Mr. Peter Gerard asked for more information concerning the junctions between the precast elements and those which were cast in situ, for it seemed to him that those junctions might be the weakest points of the structures.

M. C. Guillemin-Tarayre said M. Brice agreed that the junctions presented a somewhat difficult problem, but he did not agree that they were the weakest points

in the structures ; he emphasised that by the arrangement of suitable reinforcement the difficulties could be overcome in a satisfactory manner.

THE PRESIDENT, at the conclusion of the discussion, thanked M. Brice, Mr. Gerard and their team of experts for the most interesting way in which they had answered the questions raised. He felt sure they would be able to expand their remarks when they were able to read the questions in detail, and all would look forward to reading the published report.

Mr. J. E. SWINDLEHURST, O.B.E. (Chairman of the British Section of the Société) expressed the thanks of the Section to the President, and through him to the Council and all associated with them in the Institution of Structural Engineers, for having provided facilities for the meeting. All who had attended, he said, had learned much, as was always the case at such joint meetings.

The British Section of the Société had been particularly fortunate in the fact that lecturers such as M. Brice were prepared to come to this country and to place their experience at our complete disposal. We were most grateful to them ; indeed, one could not express one's feelings adequately in terms of gratitude, for they were much deeper. The wonderful spirit of co-operation existing between engineers had once again been exemplified by the President and his colleagues in the Institution in inviting the members of the British Section of the Société to meet again at the Institution's headquarters.

THE PRESIDENT returned thanks for the very kind remarks Mr. Swindlehurst had made, and said he would convey them to the Council.

Book Reviews

(*continued*)

Publications of the International Association for Bridge and Structural Engineering, Vol. XI, 1951, Vol. XII, 1952, and Preliminary Publication of the 4th Congress, Cambridge and London, 1952. (Zurich : Leeman.) 389 pp. (9½ in. × 6½ in.), 324 pp. (9½ in. × 6½ in.), and 925 pp. (9¾ in. × 7 in.).

These publications of the International Association for Bridge and Structural Engineering cover a large variety of original papers dealing with structural engineering in the widest sense of the term.

Volume XI contains sixteen papers, of which seven are in English, six in French, and three in German. Amongst the papers are the following : Spatial supporting frames ; Vierendeel truss analysis using equivalent systems ; Corrugated concrete shell structures ; The exact membrane theory of prismatic structures composed of thin plates ; Deformations of reinforced concrete, and a new method of increasing the carrying capacity of a strut axially loaded at the ends.

There are seventeen contributions to Volume XII, eight in English, four in French, and five in German. Papers of particular interest are on : Corrosion problems of concrete ; Shear deflections in latticed structures ; Tests concerning creep and shrinkage losses in pre-stressed concrete ; Continuous cylindrical beams of prismatic cross-section ; The sectional method in slab theory and the influence of frictional forces and bearing pressures in connection with prestressing in steel-concrete structures.

The Preliminary Publication of the Fourth Congress is divided into three sections, the first dealing with bases of calculation, safety of structures, loading, dynamic problems and deformation, and on the development of methods of calculation. Papers in the second section are on metallic structures and practical applications, and in the third section there are papers on concrete and reinforced concrete structures and current problems and on prestressed concrete. There are fifty-nine papers in all, thirty-three in English, twenty-three in French, and three in German.

In all three volumes, in order to make the matter easier to understand, the titles of the papers and summaries are given in all three languages.

Model Analysis of Structures, by T. M. Charlton. (London : E. & F. N. Spon, 1954.) 142 + xii pp., 8¾ in. × 5½ in. Price 21s.

The book deals with model analysis by two main methods : the direct, in which measured loads are applied to the structure ; and the indirect, in which influence lines are drawn by plotting the deformed shape of the structure due to a displacement applied at the point under consideration.

The Author claims that the latter method can be used in the drawing office without " the attention of a specialist " ; that " no outstanding skill is needed for making models, but great care is essential " ; and that very considerable saving in time results when a model replaces analysis by theoretical methods.

This is the first book on structural models to be published in England ; it is badly needed. A new edition will probably soon be called for, and some suggestions may not be out of place here.

It is evident that the Author is himself a skilful experimental worker, and he perhaps underestimates the difficulties which others may have when they come to make models for themselves. It would be useful to have a chapter on the technique of constructing and operating models, giving advice on overcoming the troubles which are likely to be met with. Dimensioned drawings of such items as the joints used in deforming the model structure would have been preferable to the sketches given in Figs. 15 to 20, and reference might have been made to the device of comparing the flexibility of a member having an internal joint with that of a plain member of the same size.

Careful sub-editing would bring to light places where symbols are used without explanation (e.g. *U* and *W* on page 35), and would make clearer the relationships between different sections. For example, on page 85 the reader is referred to section 5.4 for the necessity of two internal joints in a member, but no enlightenment is given in that section.

However, we now have a book, written by one who really knows what he is talking about, on a subject whose importance is gradually being realised ; it makes valuable contribution to the technical literature of structural analysis.

R. J. C.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 16th December, 1954, at 5.55 p.m., Dr. S. B. Hamilton, M.Sc., B.Sc.(Eng.), A.R.C.S., M.I.C.E., M.I.Struct.E. (President) in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

Students

Barczynski, Miroslaw, of London.
Cockain, Roy Mason, of Duffield, Derbyshire.
Coopland, Jack, of Kirkheaton, nr. Huddersfield, Yorks.
Garrett, Maxwell John, of Brentwood, Essex.
Grzeda, Aleksander, of London.
Ratcliffe, Keith Urwin, of Belper, Derbyshire.
Southgate, Roger Herbert, of Wembley, Middlesex.
Ward, George, of Coxhoe, Ferryhill, Co. Durham.

Graduates

Banerjea, Amit Kumar, B.E.(Civil) Calcutta, of Stafford.
Barry, Leslie Augustus, of London.
Bhise, Sharadchandra Sadashiv, B.E.(Civil) Bombay, of Bombay, India.
Bose, Sailendra Nath, B.E.(Civil) Calcutta, of Calcutta, India.
Buchner, Trevor Ernest, B.Sc.(Civil) Natal, of London.
Clark, Bryan Sidney James, of Grays, Essex.
Craig, Cameron, of Portsmouth.
Fisher, Norman Henry, B.E.(Civil) Liverpool, of Doncaster, Yorks.
Gibson, Peter Lovatt, of Liverpool.
Grice, John Robert, B.Sc.(Tech.) Manchester, of Wythenshawe, Manchester.
Jankowski, Zbigniew-Henry, of Cheshunt, Herts.
Khatkhate, Anant Raghunath, B.E.(Civil) Bombay, of London.
Kovar, Jiri, of Mitcham, Surrey.
Lock, Peter Howard Spencer, B.E.(Civil) Natal, of Warley, Essex.
McArdle, Brian, of Salford, Lancs.
Owen, William Trevor, of Port Talbot, Glam.
Russell, Raymond George, Birkenhead, Cheshire.
Sivaswamy, Sosale Ramaswamy Sastry, B.E.(Civil) Mysore, of London.
Terry, Arthur William, of Chislehurst, Kent.
Warner, Raymond Arthur, of Ashbourne, Derbyshire.
Warren, Stanley Edwin, of London.
Wray, Reginald George, of Salford, Lancs.
Zukas, Simon Ber, B.Sc.(Eng.) Cape Town, of London.

Associate-Members

Foster, Rodney, B.Eng. Liverpool, of Manchester.
Stankiewicz-Wisniewski, Kazimierz, of Toronto, Canada.

Associate

Kennedy, John McGregor, of Glasgow.

Member

Hook, William George, of London.

Transfers

Students to Graduates

Gabrielczyk, Ryszard January, of London.
Hunter, Kenneth, of Hayes, Middlesex.
Russoff, Barry, of London.
Young, John Frederick, of Hornchurch, Essex.

Graduates to Associate-Members

Boswell, Peter, of Stafford.
Chan Weng Chiu, of Kuala Lumpur, Malaya.
Mason, John Francis, of Borrowash, Derbyshire.

Associate-Members to Members

Basharatullah, Mohammad, of Karachi, Pakistan.
Bulling, Alec, A.M.I.C.E., of Ilford, Essex.
Carter, William George, M.B.E., B.Sc. Bristol, A.M.I.C.E., of London.
Johnson, Oliver, A.M.I.C.E., of Liverpool.
Sneddon, William Harrower, of Paisley, Renfrewshire.
Prashad, Madho, of Saharunpur, U.P., India.

Associate-Members to Retired Associate-Members

Ainsworth, Ernest, of Preston, Lancs.
Berry, George Lemuel, of Otford, Kent.
Finer, Col. Henry James, of Hermanus, Cape Province, South Africa.
Mitchell, William Arnold, L.R.I.B.A., of Buxton, Derbyshire.
Tyndale, Major (H.G.) John Altibon Guise, M.C., of Chalfont St. Giles, Bucks.

Members to Retired Members

Brooks, George, of St. Annes-on-Sea, Lancs.
Harding, Henry Malcolm, F.S.E., of London.
Heathcote, Gilbert Stanley, M.V.O., O.B.E., M.A.(Cantab), of Argentine.
Mears, Robert Peel, B.A.(Cantab), M.I.C.E., of Guildford, Surrey.
Smith, Lt.-Col. Samuel, of Bristol.
Steinberg, Herbert Edward, M.I.C.E., of East Preston, Sussex.
Tasker, Sir Robert Inigo, D.L., of Folkestone, Kent.

Re-Admission

Associate-Member

Mufti, Lt.-Col. Mohammad Iftkhar-ud-Din, of Lahore Cantonment, West Pakistan.

OBITUARY

The Council regret to announce the death of Major James Wood (Member).

RESIGNATIONS

Notification was given that the Council had accepted with regret the resignations of Francis Charles Briggs, Percy Harold Page, Charles Anthony Trayte, Harold Wallhouse (Members); Guy Cawthorpe Calvert, Dwye Meredith Evans (Retired Members); George Leslie Head, Leslie Francis Short, Cyril Douglas Williams (Associates); Allan John Arthur, Henry Cunningham, Harry Stapleton Graves, David Young Hill, Norman Lush, Frederick Edward Meyer, Edward Guy Samuel Powell (Associate-Members); Solie Jal Arceivala, Derek Wilfred Berry, Rex Edward Bond, Alexander Hughes Bowman, Desmond

CHAPMAN, Roy Oliver HIRST, Charles Wallace MATHESON, Walter Frederick MOUNTNEY, John PATTEN, Richard Finbarr Denys REED, Thomas Wood RUSSETT (Graduates); Reginald BIDGOOD, Peter Frank MEAD (Students).

EXAMINATIONS, JULY, 1954

The Examinations of the Institution will next be held at centres in the United Kingdom and Overseas on July 12th and 13th, 1954 (Graduateship), and 14th and 15th (Associate-Membership).

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, February 10th, 1954

Ordinary Meeting, 6 p.m., when Mr. F. J. Samuely, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., will give a paper on "Structural Prestressing."

Thursday, February 24th, 1955

The attention of members is particularly drawn to the fact that the following meetings will take place at the Central Hall, Westminster, S.W.1, and not at the Institution's headquarters, which is normally the case.

An Ordinary General Meeting of the Institution of Structural Engineers will be held at 5.55 p.m. This meeting is for the election of members and will be followed at 6 p.m. by a Joint Meeting with the Cement and Concrete Association and the Reinforced Concrete Association, when Professor Pier Luigi Nervi, the well-known Italian structural engineer, will give a lecture on "Some Reinforced Concrete Structures in Italy."

Owing to the large attendance expected at this meeting, arrangements have been made to reserve 200 seats for members of the Institution, and to ensure that accommodation will be available, it is suggested that members should apply to the Secretary of the Institution for reserved tickets.

As Professor Nervi's paper was not received in time for publication in the February issue of the Institution's Journal, advance copies of the talk may be obtained on application to Mr. P. H. Gooding, the Cement and Concrete Association, 52, Grosvenor Gardens, S.W.1.

Thursday, March 24th, 1955

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Dr. E. H. Bateman, M.A., B.Sc., M.I.C.E., M.I.Struct.E., A.M.I.Mech.E., A.F.R.Ae.S. (Member of Council) will give a paper on "The Development of the Elastic Theory of Continuous Frames."

Thursday, April 28th, 1955

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. G. P. Bridges, M.I.Struct.E., A.M.I.C.E., L.R.I.B.A., will give a paper on "Colliery Structures."

Members wishing to bring guests to the Ordinary and Joint Meetings referred to above are requested to apply to the Secretary for tickets of admission.

REPRESENTATION

The Council have appointed Mr. F. M. Bowen (Associate-Member of Council) to represent the Institution on the Ministry of Education Joint Committee on Higher National Certificates in Civil Engineering, in place of Dr. S. B. Hamilton, who has resigned his appointment as Institution representative.

ANNUAL DINNER

The Annual Dinner of the Institution will be held at the Dorchester Hotel on Friday, 25th March, 1955.

DRURY MEDAL AWARD

The fifth competition for the above award will take place in 1955. The subject is the design of a mobile crane.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary: envelopes to be marked in the top left-hand corner, "Drury Medal Award."

The closing date for the competition is October 1st, 1955.

The general conditions of the competition are as follows:—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say, primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution, or failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate in the presence of a Justice of the Peace or a Commissioner for Oaths.

MACLACHLAN LECTURE COMPETITION, 1955

The closing date for the receipt of entries for the next Maclachlan Lecture Competition is Thursday, March 31st, 1955. The general conditions of the competition are as follows:

1. The Institution of Structural Engineers shall institute a written lecture to be known as the Maclachlan Lecture and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Engineering as long as in every second year the subject shall be confined to steel structures. (This will be the case in 1955.)

3. Entrance into the competition for the Lecture shall be confined to Associate Members of the Institution who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning Lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award, or for any other reason, the Institution shall transfer the above sums to the Research Fund of the Institution.

Particulars of the Competition for 1955

1. The Maclachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1955.

2. The subject of the Lecture shall be confined to steel structures.

3. The work should be submitted as the script of a lecture which the author, if successful in the competition, will deliver before an audience in the course of about one hour. The development of mathematical formulæ and detailed calculations should be avoided as far as possible in the text ; if they are essential they should be embodied in appendices. Photographs, drawings, graphs, etc., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.

4. Six copies of each Lecture should be submitted and should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the Institution is Thursday, March 31st, 1955.

PAPERS FOR PUBLICATION

The Literature Committee would be glad to consider offers of papers for presentation at the Institution or for publication in the Journal.

The following is a summary of the Committee's requirements relating to articles and papers : a copy of the full conditions may be obtained from the Secretary.

(1) Articles must be of an appropriate character, having a bearing upon structural engineering or upon some kindred scientific or constructional subject, and must be approved by the Literature Committee. A short title is an advantage.

(2) Contributions must be original either in subject-matter or in presentation. Articles which have already been published or have been read to other organised bodies, or are carelessly prepared, will not be accepted for publication.

(3) The style of writing will necessarily vary with the individual, but authors are requested to write as plainly and simply as their subject will allow. Papers should be written in the third person.

(4) Where the subject allows, a brief introduction or synopsis should state clearly the purpose and scope of the paper or article, and the author's conclusions or recommendations should be summarised at the end of the paper.

In order to facilitate the indexing of articles for reference, the author will be required in addition to prepare a short precis not exceeding 25 words for inclusion under the title of the paper on the contents page of the Journal.

(5) Illustrations are desirable where they assist in explaining the context or are fundamental to the subject. They should not be used if unnecessary for these purposes. Illustrations may be either line drawings or photographs.

(6) Line drawings must be specially prepared for reproduction on smooth white paper, or clear tracing paper, with heavy main lines and large clear lettering drawn in Indian ink with a mapping pen. Alternatively, the author may submit drawings on one sheet of paper with the relevant lettering on a cover sheet of tracing paper.

The printed page of THE STRUCTURAL ENGINEER is 7 in. wide by 10 in. deep. The drawings, where practicable, should be prepared not larger than twice this size with a view to half-scale reproduction. Unavoidably large drawings which require reduction to one-third size or less, must be specially heavy and with proportionately large lettering for clear reproduction. Ordinary working drawings are not satisfactory.

(7) Where photographs are submitted they should be printed *black on glossy paper.*

(8) MS. typewritten in double spacing should be submitted in duplicate.

Brevity is an advantage and papers should not normally exceed 7,00 words in length.

EXAMINATIONS

Preparation for the Examinations of the Institution by Attendance at Technical Colleges

A candidate for Graduateship or Associate-Membership may be able to attend a technical college ; these notes are intended to guide him in choosing the most suitable instruction.

Preparation for the Graduateship Examination

Technical Colleges offer :

(a) Full-time courses for degrees or Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At technical colleges courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

Preparation for the Associate-Membership Examination

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering ; these cover only part of the requirements for the Associate-Membership Examination.

Colleges in List "A" provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

LIST "A"

Bath Technical College.
Belfast College of Technology.
Birmingham College of Technology.
Bolton Municipal Technical College.
Bradford Technical College.
Bridgend Technical College.
Chesterfield College of Technology.
Derby Technical College.
Dudley and Staffordshire Technical College.
Glasgow Royal Technical College.
City of Liverpool College of Building.
L.C.C. Brixton School of Building, S.W.4.
L.C.C. Hammersmith School of Building and Arts and Crafts, W.12.
Manchester College of Technology.
Middlesbrough, Constantine Technical College.
Nottingham and District Technical College.
Salford, Royal Technical College.
South-East London Technical College, Lewisham Way, S.E.4.
South-West Essex Technical College, Walthamstow, E.17.
Stafford, County Technical College.
Stockport College for Further Education.
Twickenham Technical College.
Willesden Technical College, N.W.10.

Colleges in List " B " provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete.

LIST " B "

Brighton Technical College.
Cardiff Technical College.
Edinburgh, Heriot-Watt College.
Huddersfield Technical College.
Leeds College of Technology.
London, Battersea Polytechnic, S.W.11.
London, Northampton Polytechnic, E.C.1.
L.C.C. Westminster Technical College, S.W.1.
Newcastle-upon-Tyne, Rutherford College of Technology.
Plymouth and Devonport Technical College.
Preston, Harris Institute.
Rotherham College of Technology.
Wigan Mining and Technical College.
Woolwich Polytechnic, S.E.18.

Students are advised to take the organised courses in Structural Engineering where these are available.

JOURNAL CASES AND BINDING, 1954

A binding case can be supplied for the twelve issues of the Journal, January-December, 1954 (Volume 32), price 11s. 6d., post free. The price for binding in half-leather, inclusive of packing and postage, is 27s. per volume.

It is requested that all parcels containing Journals forwarded for binding should bear the name and address of the sender and must be despatched to reach the Institution by the 4th April, 1955.

An Index will be included in all bound volumes. Members making their own arrangements for binding may obtain a copy of the Index upon application to the Secretary.

Copies of the Index will be provided for subscribers and those on the complimentary list.

INVITATION TO SUSSEX MEMBERS

The paper on " The Design and Construction of the Nuclear Reactor Buildings at Windscale Works, Sellafield " which Mr. D. R. R. Dick gave before the Institution on November 25th is to be repeated at a meeting of the Southern Association of The Institution of Civil Engineers to be held at Brighton Technical College on Thursday, March 24th, 1955, at 6 p.m. Members of the Institution of Structural Engineers resident in Sussex who may have been unable to hear the paper in London are cordially invited to attend this meeting and will be welcome to join members of the Association for tea at 5.30 p.m. In order that catering arrangements may be made, members who intend to be present are requested to notify the Honorary Secretary, Mr. I. Wilson, B.Sc., A.M.I.C.E., City Engineer & Surveyor's Office, " Greyfriars," Chichester, Sussex.

LONDON GRADUATES' AND STUDENTS' SECTION

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Tuesday, 15th February, 1955

Address by the President of the Institution, 6 p.m.

Tuesday, 15th March, 1955

Annual General Meeting, followed by a film display, 6 p.m.

Hon. Secretary : J. A. Pope, 53, Cranleigh Drive, Leigh-on-Sea, Essex.

INSTITUTION LIBRARY

Conditions of Loan from the Library

1. The Library is available to members of the Institution only. In special circumstances, facilities may be granted to non-members to use the Library, but they will not be allowed to borrow books.
2. Members of the Institution may borrow any book or pamphlet which is in general circulation, and also copies of Transactions and periodicals.
3. Rare books and current works of reference are not available on loan, but may be consulted in the Library.
4. Members may borrow books either by making personal application or by post.
5. The period of loan is a fortnight, but this period may be extended for a further fortnight provided that the book is not required by any other member, and provided that application for extension is made before the original period of loan has expired. The Institution pays all outgoing postage. The members are responsible for the return postage.
6. Loans are only made on the understanding that the borrower will replace any book or periodical that may be lost or damaged through any circumstance connected with the loan, including the period in transit to and from the Library. All publications should be returned by registered post and adequately protected against damage.
7. The Library is open from Monday to Friday, from 9.0 a.m. to 5.0 p.m. during the Session, and 9.3 to 5.0 p.m. during the Recess.
8. A Library Catalogue is not available at present.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Tuesday, February 1st, 1955

Joint Meeting with the Reinforced Concrete Association, at the Liverpool Engineering Society, Dale Street, Liverpool, at 6.30 p.m. Dr. D. D. Matthews, M.A., D.Eng., M.Sc.(Eng.), M.I.Struct.E., A.M.Am.Soc.C.E. (Member of Council), will give a paper on " Prestressed Concrete Bridge at Nottingham."

Wednesday, February 2nd, 1955

The above meeting will be repeated at 6.45 p.m.

Monday, February 14th, 1955

Joint Meeting with the Institution of Civil Engineers, at the College of Technology, Manchester, at 6.30 p.m., when Mr. T. C. Waters, M.I.Struct.E., will give a paper on " The Process Building at Capenhurst."

Monday, February, 28th, 1955

At the College of Technology, Manchester, at 6.30 p.m. Mr. D. R. R. Dick, B.Sc., M.I.C.E., on " The Design and Construction of the Nuclear Reactor Buildings at Windscale Works, Sellafield."

Wednesday, March 30th, 1955

Joint Meeting with the Institute of Welding (Manchester Branch), College of Technology, Manchester, at 6.30 p.m. Mr. E. Lightfoot, M.Sc.(Eng.), A.M.I.C.E., A.M.I.Struct.E., will give a paper on " Dynamic Stresses in Structures."

Thursday, April 28th, 1955

At the College of Technology, Manchester, at 6.30 p.m. Annual Business Meeting, followed by a film.

Joint Hon. Secretaries : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire ; M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford, 3, Lancs.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, February 8th, 1955

At the Supper Room, The King's Hall, Queen Street, Derby, at 7 p.m. Mr. F. Brooksbank, M.A.(Cantab.) (Graduate), on " Economics in Welding Design."

Friday, February 25th, 1955

At the Department of Civil Engineering, The University, Birmingham, at 6 p.m. Professor S. C. Redshaw, Ph.D., D.Sc., M.I.C.E., F.R.Ae.S., on " Modern Methods of Experimental Stress Analysis in Structural Engineering."

Wednesday, March 9th, 1955

At Loughborough College of Technology, Loughborough, at 7 p.m. Dr. J. M. Harvey, M.Sc., and Mr. E. Griffin, on " The Design and Manufacture of Cold Formed Steel Sections as Structural Members."

Friday, March 25th, 1955

At the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m. Mr. V. H. Lawton, M.I.Struct.E., on " Some Notes on the Intended Revisions to B.S.449."

Friday, April 22nd, 1955

Annual General Meeting, followed by Joint Meeting with the Reinforced Concrete Association, James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m. Mr. J. A. Derrington, B.Sc., D.I.C., A.M.I.C.E., A.M.I.Struct.E., will give a paper on " Recent Developments in the Technique of Precasting Concrete Structures."

Tuesday, April 26th, 1955

At the Gas Showrooms, Nottingham, at 7 p.m. Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E., M.I.Struct.E., M.Inst.W.E., M.Am.Soc.C.E., M.Cons.E. (Hon. Librarian), on " Some Unusual Industrial Structures."

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES GRADUATES' AND STUDENTS' SECTION

The following meetings have been arranged :

Wednesday, March 30th, 1955

At the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m. Mr. C. B. Brewington, B.Sc., A.M.I.C.E. (Graduate), on " Load Bearing Walls."

To be followed by Annual General Meeting.

Hon. Secretary : A. K. A. Costain, A.M.I.C.E., 134, Witherford Way, Weoley Hill, Birmingham, 29.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, February 1st, 1955

At the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, at 6.30 p.m. Mr. F. A. Partridge, B.Sc., A.C.G.I., M.I.C.E., on " The Plastic Theory."

Wednesday, February 2nd, 1955

The above meeting will be repeated at the Neville Hall, Westgate Road, Newcastle upon Tyne, at 6.30 p.m.

Tuesday, March 1st, 1955

At the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, at 6.30 p.m. Mr. M. F. Palmer, M.I.C.E., M.I.Struct.E., on " Fabrication and Erection of Steel Bridges."

Wednesday, March 2nd, 1955

The above meeting will be repeated at the Neville Hall, Westgate Road, Newcastle upon Tyne, at 6.30 p.m.

Tuesday, March 16th, 1955

At the Neville Hall, Westgate Road, Newcastle upon Tyne, at 6.30 p.m.

Ladies' Guest Night.

Wednesday, March 17th, 1955

At the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, at 6.30 p.m.

Ladies' Guest Night.

Tuesday, April 5th, 1955

Annual General Meeting at the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, at 6.30 p.m., when Mr. D. J. Leggot, A.M.I.C.E., A.M.I.Struct.E., will give a paper on " Some Aspects of Industrial Building Design."

Hon. Secretary : Captain O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, February 1st, 1955

At the College of Technology, Belfast, at 6.45 p.m. Mr. D. V. Pike, M.I.Struct.E., A.M.I.C.E., will give a paper on " Structural Uses of Aluminium."

Tuesday, March 1st, 1955

At the College of Technology, Belfast, at 6.45 p.m. Film Evening.

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., " Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Friday, February 11th, 1955

Joint Meeting with the Glasgow and West of Scotland Association of The Institution of Civil Engineers, at The Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, at 7.15 p.m., when Mr. F. A. Partridge, B.Sc.(Eng.), A.C.G.I., A.M.I.C.E., will give a paper on " The Plastic Theory as Applied to the Design of Mild Steel Structures."

Tuesday, March 29th, 1955

At the Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, at 7 p.m. Mr. D. R. Dick, B.Sc., M.I.C.E., on " Design and Construction of Nuclear Reactor Building, Windscale Works, Sellafield."

Tuesday, April 26th, 1955

At the Cadoro Restaurant, Union Street, Glasgow, at 6 p.m. Annual General Meeting.

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., " Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, February 18th, 1955

At Exeter, at 7 p.m. Mr. F. E. Somerset, on " The Efficiency of the Tube Section." (Accompanied by a film.)

Friday, March 18th, 1955

At Plymouth, at 7 p.m. Mr. D. R. R. Dick, B.Sc., M.I.C.E., on " Design and Construction of the Nuclear Reactor Building at Windscale, Cumberland." (Mr. S. G. Silhan, A.M.I.Struct.E., will read the paper on behalf of the Author.)

Joint Hon. Secretaries : E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay; C. J. Woodrow, " Elstow," Hartley Park Villas, Tavistock Road, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, February 16th, 1955

Junior Members' Evening at the Mackworth Hotel, Swansea, at 6.30 p.m.

Tuesday, March 15th, 1955

At the South Wales Institute of Engineers, Park Place, Cardiff, at 6.30 p.m. Mr. A. C. Vivian, B.Sc., A.C.G.I., M.I.C.E., M.I.Struct.E., on " Kathodic Protection of Steel Structures."

Wednesday, March 30th, 1955

At the Mackworth Hotel, Swansea, at 6.30 p.m. Major A. F. Allen, M.I.C.E., on " Description of Neath By-Pass Constructional Work." (Illustrated with lantern slides.)

Friday, April 22nd, 1955

Annual Dinner at Porthcawl. The President and Secretary of the Institution will be present.

Hon. Secretary : K. J. Stewart, A.M.I.C.E., A.M.I.Struct.E., 15, Glanmor Road, Swansea.

WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, February 4th, 1955

At the University of Bristol Geology Lecture Theatre, at 6 p.m. Mr. Clifford E. Saunders, M.I.Struct.E., will give a paper on " Some Effects of Prefabrication on Post-War Buildings."

Wednesday, February 16th, 1955

Annual Dinner at the Royal Hotel, Bristol.

Thursday, March 3rd, 1955

Combined Meeting with the Institution of Civil Engineers, in the Reception Room, University of Bristol, at 6 p.m., when Mr. W. J. Sivewright, M.A., A.M.I.C.E., and Mr. S. P. Whittington, B.Sc., A.M.I.C.E., will give a paper on " Reconstruction of Ground Floor of Cold Store, Avonmouth Docks."

Friday, April 1st, 1955

At the University of Bristol Geology Lecture Theatre, at 6 p.m. Annual General Meeting, followed by a film show.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, February 16th, 1955

At the Great Northern Hotel, Leeds, at 6.30 p.m. Mr. F. N. Sparkes, M.Sc., M.I.C.E., will give a paper on " The Principles of Quality Control."

Wednesday, March 16th, 1955

To be arranged.

Friday, March 18th, 1955

Annual Dinner and Dance, at the Parkway Hotel, Bramhope, Leeds, at 7 p.m.

Wednesday, April 20th, 1955

At the Great Northern Hotel, Leeds, at 6.30 p.m. Annual General Meeting, followed by a paper to be arranged.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., P.O. Box No. 3306, Johannesburg, South Africa.

During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone : 34-1111, Ext. 257.

Natal Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co. Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., The Reinforcing Steel Co. Ltd., P.O. Box 2962, Cape Town.

The Development of the Elastic Theory of Continuous Frames*

by E. H. Bateman, M.A., B.Sc., Ph.D., M.I.C.E., A.M.I.Mech.E. A.F.R.Ae.S.
(Member of Council)

Summary

The historical development of methods of analysis, for use in the design of continuous frames, is traced from the introduction of the theorem of three moments, almost a hundred years ago, to the contemporary application of electronic digital computers. The types of frame under consideration range from the highly redundant skeleton of a city building to the highly deformable structure of a large span single storey building or hangar. The analysis is restricted by the conventions of practical structural engineering design, namely that axial deformations of frame members may be discounted, and that columns are not so slender as to raise problems of elastic instability. The principal methods—moment-area, slope-deflection, strain-energy, moment-distribution, relaxation, etc.—are briefly summarised, and a case is presented for the need of more than one method to meet different design requirements.

Analysis of Contents

1. The historical background.
2. On the need for more than one method.
3. Basic equations and continuity relations.
   - 3.1 Beam theory.
   - 3.2 Moment-area relations.
   - 3.3 Slope-deflection equations.
   - 3.4 Comparison of basic equations.
   - 3.5 Theorem of three moments.
   - 3.6 Other continuity relations.
4. Moment-distribution.
5. Strain-energy methods.
6. Methods of computation.
   - 6.1 Relaxation.
   - 6.2 The electronic digital computer.
7. Recommended methods.

1. The historical background

It is now almost a hundred years since Clapeyron derived the Theorem of Three Moments,[1] which is the foundation stone of the elastic theory of continuous frames, so that the time is opportune for a review of progress during the past century. A continuous frame, in this context, is one which depends for stability on the rigidity of the joints and not on triangulation; it includes the "skeleton" of J. F. Baker[2]—the "ossature" of the French literature—in the sense of a cellular frame for a multi-storey building, as in Fig. 1; it also includes the polygonal arch which has common applications in single-storey buildings of large span, as in Fig. 2.

This limitation of the designation "frame" conforms with long-established practice in the German literature in which "Rahmen," the frame, is always distinguished from Fachwerktrager, the truss, as in "Der Rahmen,"[3] and "Rahmenformeln."[4] Unfortunately in English and American usage, the word "frame" has been freely applied to any structure formed of struts and ties, or beams and columns, that is of bars or members jointed at their ends and depending for stability on either triangulation or joint rigidity, or on both in common, as in "Modern Framed Structures,"[5] "Analysis of Framed Structures,"[6] and many similar titles. Horace Lamb[7] indeed went so far as to identify the frame with the triangulated structure, thus—in his elementary book "Statics":—

> "The theory of framed structures claims attention on several grounds. It has, of course, important applications in practice, in the design of roofs and bridges; whilst it appeals to the theoretical student by the interesting application of statical principles which it affords, and by the elegance of the geometrical and other methods which have been devised to meet the various problems which it presents. By a frame is meant a structure made up of rigid pieces, or 'members,' each of which has two joints connecting it with other members. In a two-dimensional frame, each joint may be conceived as consisting of a smooth cylindrical pin, fitting smoothly into holes drilled through the members it connects."

The difference in meaning of the word "frame" between 1916, the date of publication of Lamb's Statics, and the present day reflects the extent to which the elastic theory of continuous frames has been developed during the past few decades. For, although the foundation of a considerable part of the modern theory had been well established at least as early as the eighteen-nineties by Otto Mohr, following the enunciation of basic theorems for structures of both types, frames and trusses, by Lamé and Clapeyron,[8] Maxwell,[9] and Castigliano,[10] the work of Mohr, who introduced the slope-deflection equations[11] as well as the moment-area method of analysis,[12] was for many years unregarded in England and America. Arthur Morley, for example, author of one of the most widely circulated English books on Theory of Structures[13] which was first published in 1908, had not introduced the slope-deflection equations as recently as 1943; Johnson, Bryan and Turneaure mentioned the slope-deflection method in a short chapter added to the original

*Paper to be read before a meeting of the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 24th March, 1955, at 6 p.m.

" Modern Framed Structures " in the tenth edition,[5] published in 1929, and then apparently only because the method had been re-discovered in America in 1915 by G. A. Maney.[14]

Although Mohr's work was largely unknown to English and American engineers for so long, traces of his influence may be occasionally discerned, as in Andrews' " Theory of Structures "[15] and E. H. Salmon's " Materials and Structures."[16] A. E. H. Love, however, who included a short chapter on continuous beams in the third edition of his " Elasticity,"[17] took due note of the German authorities. His treatment there of continuous beams is now of course of no more than antiquarian interest, but the short passage quoted below from his well-known historical introduction gives a good picture of the state of knowledge at that time, 1920—a picture which complements the impression given by Horace Lamb in the passage quoted above. Love wrote :—

> " The problem of a continuous beam resting on several supports was at first very difficult, as a solution had to be obtained for each span by Navier's method, and the solutions compared in order to determine constants of integration. The analytical complexity was very much diminished when Clapeyron[1] noticed that the bending moments at three consecutive supports were connected by an invariable relation, but in many cases the analysis is still formidable. A method of graphical solution has, however, been invented by Mohr[18] and it has, to a great extent, superseded the calculations that were formerly conducted by means of Clapeyron's Theorem of Three Moments. Many other applications of the theory of flexure to problems of frameworks will be found in such books as those of Müller-Breslau,[19] Weyrauch[20] and Ritter[21] ; a considerable literature has sprung up on this subject, but the use made of the Theory of Elasticity is small."

Progress following Love's assessment has not, however, depended to any very great extent on the gradual percolation, diffusion and assimilation of Mohr's ideas and methods outside the frontiers of continental Germany, notwithstanding the continuing importance of Mohr's classical contributions in this field. The next advance was made in America. In 1930 Hardy Cross[22] introduced his method of Moment-Distribution, and followed his original brief monograph two years later with " Continuous Frames in Reinforced Concrete,"[23] written in collaboration with N. D. Morgan. It is difficult to overrate the significance of this contribution by Hardy Cross, for the reasons which will be developed presently, and it is therefore all the more difficult to understand the petty and perfunctory acknowledgment of it made by S. Timoshenko and D. H. Young in their " Theory of Structures "[24] in the following terms :—

> " This method (of successive approximations) was developed in connection with calculation of secondary stresses in trusses and is described in the book by O. Mohr[25] in 1906, p. 429. In U.S.A. the method was first used by S. Hardesty and is fully explained in the book by J. A. L. Waddell.[26] The extension of the method to the analysis of highly statically indeterminate framed structures is due to K. A. Calisev who used it in the analysis of building frames with and without lateral constraints,[27] 1923. The final form of the method of successive approximations was obtained in the paper by H. Cross.[28] Solutions of many engineering problems by using the method of successive approximations are discussed in the book by R. V. Southwell."[29]

Setting aside the extraordinary implication that lies on the surface of this surprising epitome, to wit that Relaxation Methods stem from the genius of Otto Mohr, one may well ask why Timoshenko and Young should skip two and a half centuries when they could have more properly commenced their comment :—

> " This method (of successive approximations) was developed in connection with the solution of algebraic and transcendental equations by I. Newton in 1672 . . ."

Sir Richard Southwell, on the other hand, in his " Elasticity,"[30] refers very briefly to " the moment-distribution method of Professor Hardy Cross, which has, in recent years, completely revolutionised the treatment of frameworks having rigid joints." This appreciation by an authority no less eminent than Stephen Timoshenko is unreservedly recommended in preference to the latter's attempted denigration. It is of course true that moment distribution has an arithmetical analogue which is manifested as the evaluation of a set of equations by a method of successive approximations. But moment distribution, like the original concept of Southwell's Relaxation, is in fact a solution by the systematic relaxation of constraints, which is completely independent of any consideration of algebraic equations. Quite apart, however, from any microscopic assessment of the details of the novel, elegant, and powerful, method introduced by Hardy Cross, it is due almost entirely to his influence that J. F. Baker[31] was able to say in November 1951, with vigorous but possibly not extravagant emphasis :—

> " During the past 20 years, greater advances have been made in the elastic stress analysis of structures than in any other branch of applied mechanics. Before that time it was possible, and then only with great labour, to deal with a structure of not more than twelve degrees of redundancy, so that the primary and secondary stresses in most practical structures could not be determined. Now there are virtually no insoluble theoretical problems . . ."

To attribute to the influence of Hardy Cross the major credit for the advances made in the two decades immediately prior to 1951 does not, of course, mean that every problem in the elastic stress analysis of structures is best attacked by the method of moment distribution. Rather, that Hardy Cross, by so effectively demolishing what was before his time a very formidable barrier, opened up new fields of investigation in addition to his own immediate contribution. Without Hardy Cross, it seems improbable that O'Sullivan in 1946 could have been writing[32] on the solution of problems not amenable to moment distribution methods. It may also be doubted whether J. F. Baker would have been able to turn so soon from his earlier work on the elastic theory of structures to the development of the plastic theory of design, whether in fact without Hardy Cross he would by now have been in a position to promise a sequel to Vol. 1 of " The Steel Skeleton."[2]

The present discussion is deliberately limited to the development of the elastic theory, but in view of recent progress in the plastic theory of design it may well be asked whether there is in fact any need to devote further attention to the elastic theory. It is eminently reasonable to suppose that, for some purposes, structures may be designed entirely without regard to elastic behaviour, but it is as yet by no means established that the elastic theory could ever be completely discounted for structures on which live loads might predominate such as very tall buildings under wind loads,[33] single storey buildings with heavy crane loads, and oper

ame (vierendeel) girders for bridgework. One impor-nt consequence of the plastic theory which is extremely levant in this context is the toll which it must take the time and attention of the structural engineer, hether as a student or in practice. For this reason one the simplification and codification of the elastic eory is urgently necessary.

Some engineers, no doubt, who have adopted a par-cular method or methods for their own special needs, ay see no need for clarification. Slope-deflection, or irtual work, or strain energy, or moment distribution, ay well be offered as a method of universal application y anyone who should deem it necessary to look further an the relaxation methods of Southwell. Most of e teachers and text-book writers in this field have nded to go nap on one particular analytical system. notable exception is Professor A. L. L. Baker, who ggests[34] that the five or six better known systems re all of about the same merit, and that the choice hould rest on individual familiarity and experience. Vith this liberal example to follow, it would be nvidious here to attempt an assessment of Baker's daptation to multi-storey frames[35] of the virtual ork system which Müller Breslau developed so success-ully for the solution of statically indeterminate trusses. he difficulty of meeting all requirements with a single ethod will be discussed presently, but, admitting a ossible necessity for plurality, no one approaching the ubject with the mind of a novice or a layman could fail o be bewildered by the abundance and apparent variety f the methods offered or advocated by the many vriters who have ventured into this field. The follow-ng list gives a representative sample of these methods, ut the list is very far from being exhaustive. Each ethod is annotated by one or more references, and here the original source is well known the appropriate eference is cited; exceptions are items 10 and 12, here the references are chosen for accessibility, and tem 8 where a modern reference is given for the method f complementary energy, which at least one authority has attributed to Clapeyron.

Short designations of some proposed methods:—

1. Beam theory.[36]
2. Macaulay's method.[37]
3. Theorem of Three Moments.[1]
4. Theorem of Four Moments.[38]
5. Theorem of Three Shears.[39]
6. Virtual Work.[9 19 35]
7. Strain Energy.[10]
8. Complementary Energy.[40]
9. Deformation Energy.[41]
10. Arch Theory.[13 15]
11. Moment-Area.[25]
12. Neutral Points.[42]
13. Focal Points.[43]
14. Characteristic Points.[44]
15. Slope-Deflection.[11 45]
16. Moment-Distribution.[22 23]
17. Moment-Balance.[46]
18. Fixation-Factors.[47]
19. Consistent Deformations.[48]
20. Relaxation.[29]
21. Remainder-Distribution.[49]
22. Stiffness-Factors.[50]
23. New Approach.[51]
24. Degree of Restraint.[52]
25. Column-Analogy.[23]

That the above list cannot be exhaustive is evident from consideration of the many additional varieties which could be obtained by permuting the possible alternative expressions for the applied loadings with the consequent moments and forces or deformations, as well as the optional varieties in relaxational proce-dures. The list will obviously continue to grow so long as editors and publishers continue to fall for every new catchword. Unfortunately it is not so easy to compile as long a list of worth-while problems solved. In respect of some of the less conventional methods enumerated, the end has however justified the means. Beaufoy, Bolton, and Wood, for example, have demon-strated their methods, items 22, 23 and 24, on non-trivial problems.

2. On the need for more than one method

In the present discussion it is not proposed to follow the example of S. A. Loveland,[53] one of the previous authors to communicate to the Institution a comparison of different methods of frame analysis. His contribu-tion, in a series of six instalments, covered a hundred pages in the Journal, small wartime pages but set in correspondingly small type. Loveland used eight methods in turn to solve a numerical example of a symmetrical rectangular portal, asymetrically loaded. The derivation of each of his solutions is longer than that of the present writer's general algebraic solution for the unsymmetrical portal frame,[54] from which any particular problem can be solved by numerical substitution. The portal frame test alone is no criterion of the value of a method of analysis, and indeed some of the more powerful methods[55 56] show to least advantage when applied to trivial problems.

Methods of analysis are only a means to an end, and that end is of course design. It is therefore first of all necessary to consider the purposes for which such methods are required. Begging the question, for the time being, of the possibility of the electronic digital computer being used as an instrument of design rather than analysis, the following four requirements need consideration:—

1. A method for the detailed check stressing of com-plete designs;
2. A method for the systematic analysis of families of structures;
3. A method for the rapid assessment of tentative designs;
4. A method of rapid check-stressing for use by engineers who have to take responsibility for the work of computers and stressmen.

There might also be added the requirement of a method or methods suitable for the class-room, the lecture-theatre and the examination-hall. Such a require-ment, however, raises wider issues and is dismissed as being beyond the terms of reference of the present discussion.

For requirement No. 1, a purely numerical method is adequate, but for requirement No. 2 it is necessary to retain at least one variable parameter in the solution, and it is desirable to have a structural solution indepen-dent of the applied loading, so that a calculation does not have to be re-started from scratch if the loading is varied. For requirement No. 3, accumulated experi-ence and access to tabulated solutions[4] are essential. In this connection, however, mention may be made of a contribution, by the present writer[57] in which the necessity was demonstrated of providing adequate stiff-ness in the horizontal members of a tall building frame

to resist wind loads. Criteria of similar type have since been proposed by A. L. Baker[58 35] and M. R. Horne.[59] For requirement No. 4, the methods applicable to No. 3 are possibly more important than fluency in approximate numerical methods.

The need for more than one method does not, however, rest primarily on the assumption of different types of requirement. It depends rather on the essential distinction between the two types of frame sketched in Figs. 1 and 2 respectively :

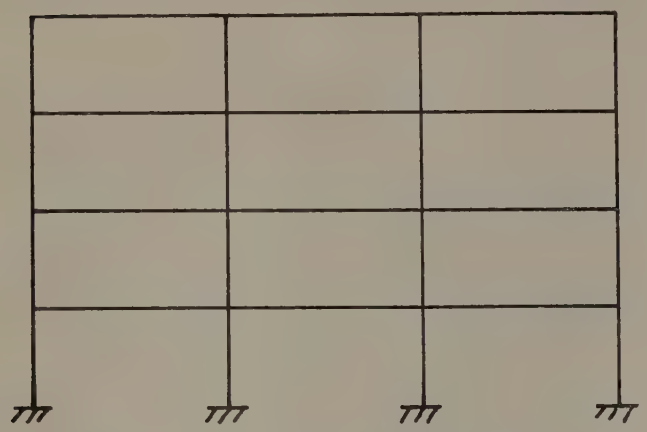


Fig. 1. The " cellular " type of frame

Fig. 2. The " linear " type of frame

The cellular type of frame (Fig. 1) is completely solved if the moments and shear forces at the ends of each member are known. There are 28 members in the structure sketched giving 112 required quantities, although these are of course not all independent, being connected by equations of statical equilibrium at the joints. The frame is also completely solved if the 14 joint rotations and the 4 possible lateral displacements are known, giving 18 required quantities. There is therefore no question here but that the cellular frame must be resolved by taking the quantities to be first evaluated as deformations rather than moments and forces. It is for this reason that the moment-distribution method or any associated system is, for such a frame, to be preferred to a strain energy method in which the indeterminate quantities are moments rather than displacements.

Consider now the linear type of frame sketched in Fig. 2. In this particular example, if the required quantities are deformations, there are 3 joint rotations, 3 joints with lateral displacements in one degree of freedom, and 10 joints with lateral displacements in two degrees of freedom, making in all 36 required quantities, though this number will be somewhat reduced by equations of compatibility of displacements. But if forces and moments are taken as the required quantities, the degree of redundancy is obviously only 6. Even with the powerful aid of the electronic digital computer, this difference between the two methods may be highly significant, for on one estimate[60] the time required for solution increases more rapidly than the cube of the number of required quantities. Thus for the linear frame, any method based on the relaxation of constraints, in which operations have to be performed on the joint rotations and displacements, must take far longer than methods of solution for moments or forces. It is not surprising therefore that Southwell in an early edition of his " Elasticity "[30] should have used this type of frame as an illustration of problems too complex for exact mathematical treatment, but for which solutions might be obtained by experiments on models. In view of the solutions[50 55] that have subsequently become available, the alternative view of J. F. Baker may be preferred that " now, there are virtually no insoluble theoretical problems,"[31] even though doubt may be reserved in some quarters on the precise interpretation of mathematical insolubility.

The distinction between the cellular and linear types of frame was first pointed out by the present writer[41] in 1937, on the only previous occasion of his presenting a paper to the Institution. This important observation made no impression on current opinion, but until the distinction is recognised there can be no hope of order emerging from the existing state of confusion. In making this plea, the author is asking for no more than recognition of a manifestation, in this field, of the classical principle of duality which every schoolboy associates with the names of Pascal and Brianchon.

3. Basic equations and continuity relations

3.1 *Beam theory.* The first attempts to analyse rigid frames, starting with the uniform continuous beam, were made by direct application of the fundamental equations of flexure, which relate slope, bending moment, shear-force, and load-intensity, respectively, to the first, second, third, and fourth derivatives of the curve of deflection. As Love[17] pointed out many years ago, in the passage already quoted from the Historical Introduction to his Elasticity, the method had in his day been long since discarded as a practicable system of frame analysis, by reason of the burden of computing and manipulating the constants of integration. The so-called method of W. H. Macaulay,[37] which is of course no more than a notational convention, slightly alleviates the burden. But even with Macaulay's method, the only plausible reason for using the beam theory to solve actual problems in frame analysis would be to demonstrate its relative crudity and ineffectiveness. Nevertheless, the beam theory remains of abiding importance as the source of all the more powerful methods of analysis.

3.2 *Moment-area relations.* The advantage of the moment-area method is that the constants of integration are eliminated at the outset by the choice of coordinates, provided the relations are expressed graphically. Furthermore, in a general analytical statement of the moment-area relations, when the constants of integration cannot be eliminated without unwarrantable changes of origin, they appear as physical entities, the slopes and deflections at particular points. By contrast, in applications of the beam theory, the constants of integration can only be interpreted as combinations of these physical elements. In the usual graphical method of presentation, in terms of the bending moment diagram for a uniform beam, the two moment-area relations are :—

1. The difference between the slopes at two points A and B is the area of the bending moment diagram between A and B, divided by the flexural rigidity, EI.
2. The deflection at B, measured from the tangent at A, is the moment about B of the area of the bending moment diagram, divided by the flexural rigidity, EI.

The method is easily applied to members of non-uniform section by taking the area or moment of area of a reduced bending moment diagram, which is derived by factoring the bending moment diagram from point to point by $1/EI$, the reciprocal of the flexural rigidity. The moment-area system, however, is not suitable for determining the statically redundant quantities in a continuous frame, because of the difficulty of graphical representation of quantities which cannot be given numerical values. Nevertheless, an understanding of the method is essential, because it is so often the best way of finding slopes and deflections when the statically indeterminate quantities have been evaluated by a more powerful method.

3.3 *The slope-deflection equations.* These equations, like the moment-area relations, express the moments at the ends of a member in terms of displacements and some function of the applied loading ; or, alternatively, the slopes at the ends of the member in terms of lateral displacements and applied loading. As in the moment-area method, two equations are required for each member, but attention is now concentrated on the absolute displacements of the ends of the member, instead of on the displacement of one end relative to the other. Thus the information conveyed by these equations appears in a more suitable form for easy transition from one member of a frame to the next. A second and very significant advantage of the slope-deflection equations is that the applied loading is now represented by fixed-end moments instead of by loading terms that have no such physical significance.

3.4 *Comparison of basic equations.* Summing up so far, the distinction between the three primary methods of describing the relation between load and flexure in a frame member may perhaps be most concisely and clearly expressed by stating the respective master equations.

The beam theory equations comprise four relations which can be expressed in a single equation as follows :

$$\left\{\theta, M, S, p\right\} = \left\{\frac{dy}{dx}, EI\frac{d^2y}{dx^2}, EI\frac{d^3y}{dx^3}, EI\frac{d^4y}{dx^4}\right\} \quad (1)$$

where the first, second, third, and fourth terms on each side of the equation are respectively equal one to the other, and the sign convention is chosen to make all term positive.

The moment-area relations are derived from the first and second of the four relations comprised in equation (1), and with the same sign convention they take the form

$$\theta_A - \theta_B = \frac{l}{2EI}\left(M_A + M_B\right) + \int_0^l \frac{m dx}{EI} \quad . \quad (2a)$$

$$y_B - y_A = l\theta_A + \frac{l^2}{6EI}\left(2M_A + M_B\right) + \int_0^l \frac{mx dx}{EI} \quad (2b)$$

where the suffixes A and B refer to the points A and B, representing the ends of the member, and m is the bending moment at any point between A and B due to the applied loading, of intensity p, acting on a span AB simply supported at A and B.

The slope-deflection equations express the end-moments in terms of slope and deflection at A and B, thus

$$M_A = F_A - \frac{2EI}{l}\left(2\theta_B + \theta_A - 3\varnothing\right) \quad . \quad . \quad (3a)$$

$$M_B = F_B + \frac{2EI}{l}\left(2\theta_B + \theta_A - 3\varnothing\right) \quad . \quad . \quad (3b)$$

where $\varnothing$ is the angular rotation of the line connecting A and B, i.e.

$$\varnothing = (y_B - y_A)/l$$

and F_A and F_B are the fixed-end bending moments at A and B. These equations are easily derived from equations (2) by eliminating M_A and M_B in turn, noting that

$$\int_0^l m dx = -\frac{l}{2}(F_A + F_B)$$

. . . . . (3c)

$$\int_0^l mx dx = -\frac{l^2}{6}(2F_A + F_B)$$

when EI is constant, which limits the application of equations (3) to the member of uniform section.

Equations (3) are easily transposed into the alternative form

$$\theta_A = \varnothing - \frac{l}{6EI}\left\{2(M_A - F_A) + (M_B - F_B)\right\} \quad (4a)$$

$$\theta_B = \varnothing + \frac{l}{6EI}\left\{2(M_B - F_B) + (M_A - F_A)\right\} \quad (4b)$$

which are suitable for use with equations of compatibility, in contrast with equations (3) which are required to combine with equations of equilibrium at the joints of a frame.

In proceeding from the beam equation to the moment-area and slope-deflection equations, some of the basic information which they convey has to be discarded, but this merely increases their efficiency for the particular purpose for which they have been developed, which is the analysis of continuous rigid frames. The moment-area and the slope-deflection equations both contain the same amount of information, but in the latter it is presented in a more suitable form, in that explicit expressions are given for the indeterminate quantities M or θ, and the applied loading is represented by fixed-end moments.

3.5 *Theorem of Three Moments.* If now consideration is given to any two consecutive spans, say AB and BC, in a continuous beam, the angle θ at B can be expressed in two different ways, according to whether it is taken as the rotation of the member AB or the member BC. Equations (4b) and (4a), with the appropriate change of suffixes in the latter, give

$$\theta_B = \varnothing_1 + \frac{l_1}{6EI_1}\left\{2(M_{BA} - F_{BA}) + (M_{AB} - F_{AB})\right\}$$

$$\theta_B = \varnothing_2 - \frac{l_2}{6EI_2}\left\{2(M_{BC} - F_{BC}) + (M_{CB} - F_{CB})\right\}$$

where the suffixes 1 and 2 denote the spans AB and

BC respectively, and $\varnothing_1 = (y_B - y_A)/l_1, \qquad \varnothing_2 = (y_C - y_B)/l_2$

Eliminating θ_B and writing M_B for M_{BA} and M_{BC}

$$\frac{l_1}{6EI_1}(M_A - F_{AB}) + \frac{2l_1}{6EI_1}(M_B - F_{BA}) + \frac{2l_2}{6EI_2}(M_B - F_{BC}) + \frac{l_2}{6EI_2}(M_C - F_{CB}) = \varnothing_2 - \varnothing_1 \quad . \quad . \quad (5a)$$

which is the equation of three moments for a continuous beam on supports at different levels, with flexural rigidities which change from span to span, but which are uniform within each span.

For supports at the same level and the same flexural rigidity in each span, equation (5a) becomes

$$l_1 M_A + 2l_1 M_B + 2l_2 M_B + l_2 M_C = l_1 F_{AB} + 2l_1 F_{BA} + 2l_2 F_{BC} + l_2 F_{CB} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5b)$$

which is not only more symmetrical and logical, but also more general than the usual presentations of this theorem. It is indeed a matter for comment that text-book writers should find so many different ways of expressing this theorem, when the use of fixed-end moments to represent the applied loads has such significant advantages.

3.6 *Other continuity relations.* If more than two members meet at a panel point, the equation of three moments no longer holds without modification, and the corresponding continuity relation becomes one of four or more moments, as in Bleich's Viermomentensatz.[38] Similarly there could be stated theorems of three or more angles or three or more shears.[39] Furthermore, the effects of applied loads can be expressed in many different ways—as, for example, in terms of the positions of the points of contraflexure in a fixed-ended beam, by the end moments when the beam is fixed at one end and pinned at the other, or by the end angles in a freely supported beam. If the different ways in which it is possible to represent the statically-indeterminate quantities are then permutated with the different ways of accounting for the applied loading, there is an astronomically large number of potential methods of frame analysis, even before including the possible methods of solving the ensuing equations. Some such special methods may conceivably be useful for the analysis of particular types of structure, but in general the distinction between them is trivial and their fundamental contribution to the theory of elastic stress analysis negligible.

4. Moment distribution

The method is now so well known that it is unnecessary here to do more than derive the rules for the two basic operations : the carry-over principle, and the distribution of the unbalanced moment at a joint. Referring to the slope-deflection equations (3), a change in θ_A, leaving θ_B and $\varnothing$ unchanged, affects M_A and M_B as follows :—

$$\frac{\partial M_A}{\partial \theta_A} = -4EK, \qquad \text{and} \qquad \frac{\partial M_B}{\partial \theta_A} = 2EK$$

where K is the stiffness I/l. Hence for a finite rotation $\delta\theta$

$$\delta M_A = -4EK\delta\theta, \text{ and } \delta M_B = 2EK\delta\theta = -\frac{1}{2}\delta M_A \quad (6a)$$

Thus, if the joint A is released, the bending moment at B is increased by half the moment taken away at A, which is the carry-over theorem, with the bending moment sign convention.

Now, if several members, denoted 1, 2, . . . n, meet at A, the release or partial release of the joint A, by rotating each member through an angle $\delta\theta$ affects the bending moments at A as follows :—

$$\delta M_{A1} = -4E_1 K_1 \delta\theta; \quad \delta M_{A2} = 4E_2 K_2 \delta\theta, \text{ etc.},$$

$$\text{Hence} \quad \frac{\delta M_{A1}}{E_1 K_1} = \frac{\delta M_{A2}}{E_2 K_2} = . \; . \; . = \frac{\Sigma \delta M_{Ai}}{\Sigma E_i K_i} \quad (6b)$$

so that an unbalanced moment at a joint is distributed among the members meeting at that joint in proportion to the stiffnesses of the several members.

It is common practice, following the original Hardy Cross system, to distribute the exact total of the unbalanced moment at each joint, starting with the unbalanced fixed-end moments, and with this regular and orderly procedure a satisfactory solution is usually reached in about 6 or 7 stages, each of which comprises a distribution and the associated carry-over. The rate of convergence can however be improved considerably if the moments distributed at each stage are 20 or 30 per cent. greater than the exact out-of-balance moments, since this increase tends to anticipate the effect of the carry-over contribution. With this simple variation of the standard procedure, very close approximations can often be obtained with only two stages of distribution and carry-over ; this method is recommended in preference to the more complicated devices, for improving convergence, which depend on special combinations of the stiffnesses of adjacent members.

The outstanding merit of the moment distribution system, apart from its obvious simplicity and ease of comprehension and assimilation, is the rapidity with which solutions can be obtained when there is no sidesway. In such cases the solution can often be written down, by moment distribution, in less time than would be required to formulate a set of moment-area or slope-deflection equations. When, on the other hand, the effect of sidesway is considerable, the elementary moment distribution procedure is not so conspicuously successful. For such cases a number of investigators, J. A. L. Matheson[61] for example, have given special attention to the development of more efficient procedures.

A further limitation of the method is that it becomes extremely complicated in applications to the linear type of frame (Fig. 2), because, in that case, the number of unknown displacements is so much larger than the number of mechanical redundancies. The interesting paradox of the moment distribution system is that, in the mathematical analogue, the unknown quantities are not moments but angles and lateral displacements. The method should therefore not be applied to problems which can be solved so much more efficiently by taking the unknown quantities as forces and moments, instead of rotations and displacements.

Since moment distribution leads to an iterative process of computation, it is not to be recommended for the systematic analysis of associated families of frames, when it is necessary to retain one or more algebraic parameters through all the stages of a calculation. Notwithstanding these limitations, moment distribution must be recognised as the most significant individual contribution to the theory of elastic analysis of continuous frames.

(*continued at head of next column*)

5. Strain-energy methods

The strain-energy in an element of a loaded beam is $\frac{M^2}{2EI}dx$ in the well recognised conventional notation.

Hence the total strain-energy U, in a straight member of uniform section can be written

$$2EI.U = \int_0^l M^2 dx \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

where $M = M_A\left(1-\frac{x}{l}\right) + M_B\frac{x}{l} + m$

if m is the bending moment due to the applied loads acting on the member when simply supported, so that m here has the same meaning as in equations (2). Now, on substitution in equation (7),

$$2EI.U =$$

$$M_A^2\int_0^l\left(1-\frac{x}{l}\right)^2dx + 2M_AM_B\int_0^l\frac{x}{l}\left(1-\frac{x}{l}\right)dx + M_B^2\int_0^l\frac{x^2}{l^2}dx + 2M_A\int_0^l m\left(1-\frac{x}{l}\right)dx + 2M_B\int_0^l m\frac{x}{l}dx + \int_0^l m^2dx$$

and since, from equations (3c),

$$\int_0^l m\frac{x}{l}dx = -\frac{l}{6}(2F_A + F_A)$$

$$\int_0^l m\left(1-\frac{x}{l}\right)dx = -\frac{l}{6}(F_A + 2F_B)$$

the strain energy can be expressed in the form

$$6EK.U =$$

$$(M_A-F_A)^2+(M_A-F_A)(M_B-F_B)+(M_B-F_B)^2+C \quad (8)$$

where K is the stiffness, I/l, and C is independent of M_A and M_B so that it does not appear in partial derivations of U with respect to M_A and M_B.

With the strain-energy available in this form, which was first stated over 20 years ago,[62] the application of Castigliano's principle to continuous frames is very greatly simplified in comparison with the conventional treatments.[63 64] As a result of this innovation, the strain-energy method is extremely efficient in application to some particular tyeps of structure.[41 54 57 65] The strain-energy method is however still generally condemned by the text-book writers in view of the heaviness of the earlier treatments.[63 64]

The advantage of the strain-energy system, as compared with the methods of slope-deflection or moment-distribution, is that, even in the most complicated conditions of sidesway, solutions are obtained without any considerations of rotations and displacements. The penalty for this very substantial simplification is that the resulting equations may be not so easy to solve by relaxational methods, as are those equations which ensue from primitive applications of the slope-deflection method. The problem of so-called "ill-conditioned" equations arising when a problem is solved in terms of forces, instead of displacements, was a subject of enlightening comment by Sir Richard Southwell[66] in his inaugural address to a meeting of the Colston Society at Bristol in 1949. But there is a very great difference between equations which give slowly converging solutions under methods of either iteration or relaxation and those highly sensitive equations which give completely false solutions when the residuals have been reduced to extremely small values. A classic example of a set of four ill-conditioned equations has been given by Southwell,[67] in his 1948 James Forrest Lecture, with an acknowledgment to T. S. Wilson. Equations of this type, as Southwell himself suggested in that lecture, are most unlikely to arise in any analysis of a practical engineering structure.

Although the strain-energy method, based on the application of equation (8), cannot in general be applied efficiently to frames of cellular type, the advantage of an energy method, in avoiding equations of compatibility or equilibrium, can be retained if the energy of deformation of a frame member is expressed in terms of rotations and displacements as follows[41] :

$$W = 2EK\left\{(\theta_A-\varnothing)^2+(\theta_A-\varnothing)(\theta_B-\varnothing)+(\theta_B-\varnothing)^2\right\} + F_A(\theta_A-\varnothing) + F_B(\theta_B-\varnothing) + C^1 \quad . \quad . \quad (9)$$

The application of this expression by taking partial derivatives with respect to θ and $\varnothing$ leads to the same result, but much more directly, as the use of the slope-deflection equations, and a solution by iteration follows the numerical pattern of a solution by moment-distribution, given the appropriate routine of iteration.[41]

6. Methods of computation

6.1 *Relaxation.* Twenty years ago the language of applied mathematics, engineering science, and physics, was enlarged and enriched by Sir Richard Southwell's introduction of the word " relaxation."[68] During these past two decades the scope of relaxation methods has been continually expanded, and their power increased, by Sir Richard and his many assistants, associates and disciples; the relevant literature is already extensive[29 69 70 71]; relaxation has now found its place in the syllabus in most engineering schools, and in some colleges it forms a subject of post-graduate vacation courses. The development of relaxation methods has generally been presented to the engineering world in the form of particular solutions of problems that had formerly been thought intractable, with the consequence that the anatomical details of the relaxation process have more often than not been masked by the glamour and brilliance of the solutions. As F. S. Shaw has written in the preface of his recent book,[70] " despite the many papers now available, there exists still a general lack of knowledge of the manipulative devices actually used in obtaining solutions."

Southwell himself has called relaxation " a mathematics for engineering science "[72] and " a particular brand of computational technique."[73] Shaw[70] goes into a little more detail when he writes : " In the ultimate, relaxation methods consist of a collection of numerical methods of solving, approximately, large numbers of simultaneous equations." The solution of a problem by relaxation therefore entails the mathematical process of expressing the physical problem as a set of simultaneous equations, sometimes as many as 500 in number,[73] together with the arithmetical or computational process of solving these equations. In the present context we are only concerned with linear algebraic equations. This is the field in which the process of relaxation was first demonstrated, but not the one in which the " relaxationist "[73] or the " relaxer "[71] has achieved the most conspicuous successes, for it is in the solution of problems which lead to partial differential equations that relaxation attains the higher orders of merit.

For the analysis of frames of the kind discussed in this paper, relaxation offers a method of obtaining solutions of the governing equations. If these equations are so formed that the " residuals " of relaxation terminology are the unbalanced fixed-end moments postulated by Hardy Cross, and if the relaxation operators correspond to the rotations and displacements of the joints of the frame, the process of relaxation will tend to follow that of moment distribution. The numerical details of the respective computations will not necessarily be identical, since relaxation does not incorporate fixed rules of procedure comparable with the primitive conventions of Hardy Cross. For, although the process was originally described as systematic,[68] the latest definition, that of D. N. de G. Allen,[71] runs as follows : " The relaxation method is far from being a method carried out according to any set rules—instead it could more accurately be described as a method of breaking such rules as are necessary formally to explain it. The good relaxer is the man who knows, by experience, which rule to break, when to break it, and by how much to break it." Shaw does not attempt such an explicit, if indirect or Irishlike, definition ; " the basic ideas," he says, " are few and easily grasped " . . . " for manipulative facility, practice is essential," and " any device is legitimate which is of assistance in obtaining a solution."

In this connexion it is relevant here to note that ad-hoc methods for improving convergence in numerical solutions of rigid frames have long been on record, as for example the present writer's remainder distribution[41 49] introduced in 1936, and the earlier contribution of Goldberg[74] published in 1933. These methods would today be properly classed as conventional exercises in relaxation, including " block " relaxation, though Goldberg at least could not have designated his method correctly two years before the publication of Southwell's original monograph.[68] Structural engineers should not therefore expect the mere mention of the word " relaxation " to remove all the difficulties which they may have met in other approaches to the problem of the frame, and the proponents of relaxation cannot wish to commend their methods on the basis of possible contributions in a relatively narrow field that has already been intensively cultivated. In this connection it is interesting to find that in the preprint of a contemporary lecture[75] moment distribution is described as " a particular case " of relaxation. This is indeed surprising in view of Sir Richard Southwell's lucid exposition[73] of the distinction between his own work and that of his predecessors in related fields of interest.

6.2 *The electronic digital computer.* Electronic digital computers have been described by B. V. Bowden, in his illuminating book " Faster than Thought "[76] as " very complicated machines that amaze us all—that can add up a column of 500 numbers while a mathematician pronounces the word ' addition '—that can compute the sines of a dozen angles to twelve decimal places while he mutters the slightly longer word ' abacadrabra '—that do more arithmetic than a man can do in a day ' while one with moderate haste might tell a hundred '."

No structural engineer, who has ever laboured for love or money to obtain the numerical solution of a problem of theoretical interest or one arising in practical design, can fail to appreciate the " new look " which this new invention has given to the prospect of outstanding developments in structural analysis. As D. Williams has written, discussing the application of " the Machine " in another context,[60] " for every kind of structure used in mechanical and civil engineering, the change which the machine can bring about can only be described as revolutionary."

But given the machine, for which we are indebted to the insight, skill, and ingenuity of all the mathematicians, physicists and electronics engineers who have contributed to its development, there is still a price to be paid for any service which it is within the capacity of the machine to render. For the electronic digital computer is universal in its potential applications, unlike the electrical analogues that are specially designed to solve particular classes of problems. In order to apply the machine to any special task, it must be given a sequence of precise instructions as to how the initial data of a posed problem is required to be manipulated. The sequence of instructions is called a " programme." For simple operations, such as taking the square root of a number, the instructions may be relatively few and the programme correspondingly short, but the construction of the simplest programme is a highly skilled operation demanding experience in the application and selection of mathematical methods, economical and logical planning, a complete understanding of the way in which the machine is designed to function, and fluency in the particular code in which both the instructions and the data have to be expressed for interpretation to the machine. Fortunately, the

ability to construct a programme does not also require a full understanding of the electronic circuits of the machine. The construction of a programme for the solution of a problem that goes beyond the tabulation of simple functions, or the mechanisation of trivial arithmetical processes, demands considerable ability in conventional mathematics and another quality as well. This is the special branch of mathematical logic now being developed and cultivated by the programme constructors who are extending the range of problems which these new machines can be made to solve.

Once a programme has been constructed for a particular class of problem, an indefinite number of numerical solutions can be obtained for all the different numerical problems in that particular class. Thus, given a programme for extracting square roots, the square root of any number can be obtained. So, given a programme for solving a structural engineers' frame, a numerical solution for any frame can be found by including, in the instructions to the machine, the geometrical data which defines the shape of the frame and the physical data which defines the strength and stiffness of the frame members.

The programme is therefore seen to comprise two parts : a " master programme " which controls the types and sequence of operations to be made on the initial data, and a " supplementary programme " which is the appropriate mathematical representation of the initial data, expressed numerically in the code of the machine. The master programme demands for its construction all those special qualities which have just been mentioned ; but, once the master programme is available, the construction of a particular supplementary programme is much more a routine matter. It may indeed be found to be a task within the capacity of the average structural engineer, whose special qualities and training, for practical design and building construction, are of a different order from those required in a fabricator of master programmes. In any case the construction of a supplementary programme is unlikely to present the structural engineer with greater difficulty than the comprehension of some of the methods of frame analysis that have already been offered for his use in solving problems of moderate complexity.

Until recently, therefore, the main obstacle confronting structural engineers, who have been looking hopefully to the prospect of using the electronic digital computer, has been the lack of a master programme. That obstacle has now been removed, at any rate in part, and the members of the Institution have every reason to be grateful to Dr. R. K. Livesley[56 77 78 79 80] for having applied his skill, as a programme constructor, to the particular class of problem in which so many of them are interested, or indeed committed in the execution of their professional responsibilities. In making this acknowledgment, reference should also be made to the part played by Professor J. A. L. Matheson in sponsoring this work in Manchester University.

It would be out of place here to attempt a detailed account of this new and important contribution to structural analysis, since the purpose of this survey has been to give a general synoptic picture of the development of methods of analysis in the perspective of an historical background. Moreover, any facts and figures about the speed and capacity of the machine, when used in this context, might be open to misinterpretation without full supporting information as to how the machine can actually be used in structural engineering practice. For these reasons, further comment is withheld until the opportunity arises for a full discussion at an Institution meeting on a paper which will no doubt before long be forthcoming from a specialist in the application of the machine to structural problems. The suggestion that the obstacle to the use of the machine by structural engineers may as yet have only been removed in part is merely to reserve the opinion that one master programme may not necessarily be the best for all problems of frame analysis.

7. Recommended methods

The commendation of the electronic digital computer in the preceding section is not intended to imply any suggestion that the design of continuous frames, or indeed of any engineering structure, can ever be made a fully automatic mechanical operation. Hopes have of course been raised that, given the geometry of a structural engineers' frame, the machine could be programmed to print out a list of the scantlings of all the frame members, so selected as to give an unique design of minimum weight. Begging the question of whether such a result might eventually be brought about by the design of machines of even higher speed and greater capacity, and by unpredictable improvements in the art of constructing programmes, the present position is that engineering design is beyond the capacity of the machine. Dr. Livesley's opinion on this point will be welcomed by every engineer with professional design experience, as confirming the view that art, inspiration, and experience are at least as important as mathematics and mechanical computation. This opinion, in summary form, is as follows :[80]

" We might argue of course that machines must be improved so that they can carry out the design work for which they appear at present unsuitable. A better attitude, however, is to regard the digital computer as essentially complementary to the human designer . . . There seems to be no point in forcing machines to carry out tasks for which human beings are much better equipped."

Here is authority enough, if authority be deemed necessary, for the proposition outlined in Section 2 that design can never be a self-contained operation. Recapitulating briefly, methods may be required for the following :—

(1) Detailed stress analysis of a particular structure ;
(2) Systematic analysis of families of structures ;
(3) Rapid assessment of tentative designs ;
(4) Spot checking of completed designs.

In view of the large number of methods already proposed by the many contributors to the literature, of which a representative sample has been reviewed in Section 1, it is obviously impossible to submit non-controversial recommendations. Attempting to answer the question without personal bias or prejudice, the present writer reaffirms the importance of distinguishing between the " cellular " and " linear " types of frame, and makes the following suggestions :

(1) For cellular frames : moment distribution, or a " machine " programme in which the statically indeterminate quantities are the joint-rotations and displacements. For linear frames : analysis by strain-energy or any other system in which the statically indeterminate quantities are moments or forces rather than displacements, with " machine " computation subject to a suitable master programme.

(2) Any method by which it may be possible, as the present author has found it to be with the strain-energy method, to retain at least one variable parameter in the solution.

(3) Assessment of tentative designs will continue to depend mainly on experience. The tabulation of known results, such as the ready-to-use formulae of Kleinlogel,[4] is of great value, and the digital computing machine offers the prospect of the range of tabulated solutions being very usefully extended.

(4) For the spot-checking of completed designs, experience is necessary and tabulated solutions are helpful. The prime need here, however, is for the establishment of simple rules prescribing permissible limits for the relative stiffnesses of associated frame-members, such as the beams and columns in a cellular type of frame.

In conclusion, the author ventures to hope that this survey will be of some assistance to potential workers in this field who wish to direct their activities into the most profitable channels, and that it may also serve as a guide to the practising structural engineer through the labyrinth of extant theories.

* * *

Acknowledgment.—The author is grateful to the Director, Royal Aircraft Establishment, for having afforded him the opportunity to write this paper, the subject of which is not on the main line of his current duty in Structures Dept., R.A.E. ; and he is further indebted to the Chief Scientist, Ministry of Supply, for permission to present the paper.

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Note.—The numbers in parenthesis at the end of several earlier references, e.g. references (20) and (21) refer to the authority, e.g. (17), for necessary citations which have not been personally verified by the present writer.

Book Reviews

Theory and Practice of Structural Design applied to Reinforced Concrete, by B. Eriksen. (London: Concrete Publications, 1953.) 9 in. × 6 in., 402 pp., 252 illus., 38 tables. Price 25s.

Primarily a student's text-book, it is, nevertheless, a book for the office library where the young Graduate, who has turned his face definitely towards specialisation in reinforced concrete construction, will find much to guide him. Here the Theory of Structures is applied to one medium exclusively and the two are married. The reader is dealing only with what is generically known as "reinforced concrete" and the mistake, so often committed, of trying to cover two media—structural steel as well as reinforced concrete in one book is avoided.

It may be said that the Theory of Structures *is* The Theory of Structures and the same principles apply whether you are designing in concrete, steel or chewing gum. That is so, but I feel I am not abnormal in liking a book which applies the Theory to one method of building at a time and that, in reading it, I do not have to visualise helical binding on one page and steel stanchions on the next (and I do not like chewing gum anyhow).

Again, it is a book to reassure the timorous, in that things are clearly explained and the "post and beam" practice still beloved of many because of their shaky mathematics, can be confidently abandoned in favour of an approach to real reinforced concrete design.

I should like to see a further volume on the same general principles but dealing with prestressed concrete and "skin" design.

R. J. V.

Industrial Electronics, by R. Kretzmann. (Philips Technical Library. London: Cleaver-Hume Press, 1953.) 234 pp., 9 in. × 6 in. Price 25s.

This latest addition to the Philips Technical Library maintains the extremely high reputation of their publications.

The text is divided into two parts, the first entitled "The tubes and their basic circuits" contains full descriptions of the various types of tubes. The second and larger part covers many of their uses for Industrial purposes. The applications of electronics are numerous, particularly in Mechanical Engineering; however, the Structural Engineer will find much information of value, especially in the section devoted to electronic control of welding operations. It was disappointing to find that in Chapter 11 of Part 2 on "Electronic Counting Circuits," no reference was made to the latest development in this field, namely the dekatron.

Presentation of the subject matter, the many excellent diagrams and photographs, are to be commended. The book will be of interest and value to both practising engineers and students.

D. M. B.

Solution of Problems in Engineering Drawing and Projective Geometry, by K. L. Jackson, M.Sc. (London: Pitman, 1954.) 52 + vii pp., 11 in. × 8½ in. Price 10s.

This book contains a selection of problems in engineering drawing and projective geometry specially chosen from examination papers of the University of London for the use of students preparing for the B.Sc.(Eng.) degree or taking National Certificate courses.

Part I contains eleven problems on engineering drawing most of which are reprints of University College examination questions, the remainder being General Internal and External questions. Solutions to these questions are given in the form of eleven printed sets of drawings.

Part II likewise reprints twenty-four questions from University College, Imperial College, King's College, and Internal papers which are in turn followed by complete solutions.

While it is acknowledged that all these questions are adequately explained and will be helpful to students for passing university examinations, most of the book is taken up with printed solutions. No doubt this can be of great assistance to the overloaded teaching staff, but one cannot help wondering whether students really learn the work fully when they are able to copy out worked solutions. They can produce lovely drawings and probably the conscientious students are able to work out their own salvation; but this book, particularly Part II, could have been of much greater value to all learners if it had printed double the amount of questions and had managed to just hint at the solutions.

H. P. S.

(*continued on page 95*)

Critical Loads of Tall Building Frames

By W. Merchant, M.A., A.M.I.C.E., A.M.I.Struct.E.

Synopsis

In a previous paper[1] we have pointed out the desirability of being able to calculate the critical load of building frames. This paper shows how the sway critical load of any symmetrical single bay frame may be obtained. It also gives the results of numerical calculations for a particular case and an extension to many bay frames.

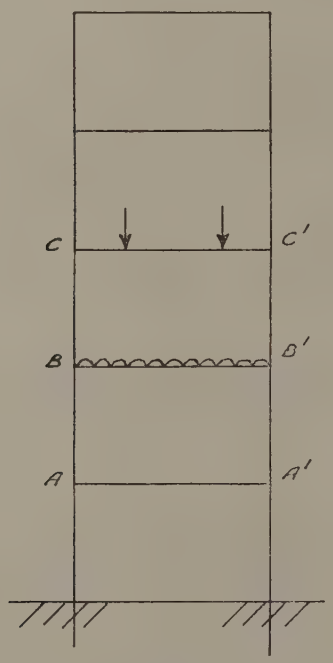


Fig. 1

Symmetrical Single Bay Rigid Frames

The type of frame we are considering is shown in Fig. 1. The floors may be of different heights and the beam and stanchion sections may change from floor to floor, but the two stanchions are identical and the frame is symmetrically loaded.

The problem we are considering is for a particular loading pattern what will be the lowest value of the loading parameter at which the structure will become unstable if all the members remain elastic.

The lowest critical mode of such structures is the fundamental cantilever one, due to sidesway, characterised by equal rotations of the joints in the two stanchions.

When stability has to be taken into account the relations between the end moments in a member and the rotations and displacements of the joints differ from those given by the usual Slope Deflection equations. The important relations are summarised in Fig. 2. In that figure s is the non-dimensional stiffness coefficient and c the non-dimensional carry over coefficient. Both s and c are functions of P/P_E where P is the axial load in the member and P_E the Euler load, i.e. the lowest critical load of the member if acting as an isolated pin ended strut. Graphs of s and c are given in the paper previously referred to.[1]

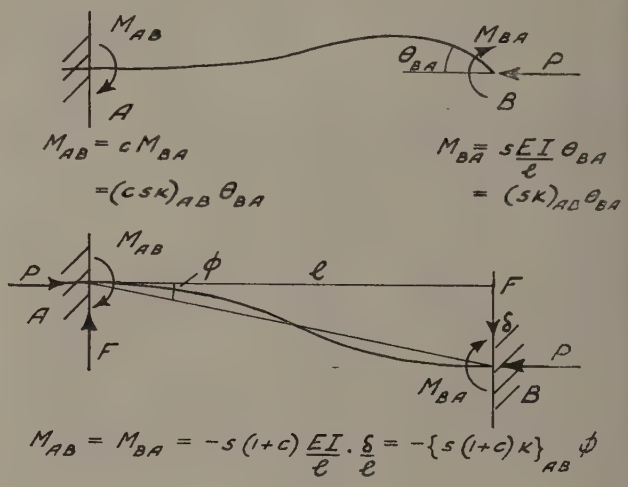


Fig. 2

If P/P_E is small $s \to 4$ and $c \to \frac{1}{2}$ and the relations of Fig. 2 become, in agreement with the usual Slope Deflection equations which are applicable when stability effects can be ignored.

Suppose now that floor B sways through an angle $\varnothing$ relative to floor A, all the joints being prevented from rotating. The moments in the stanchion length AB are then as shown in the lower diagram of Fig. 2.

Considering the equilibrium of member AB

$$2s(1+c)k\varnothing = Fl + P\delta$$
$$= Fl + \frac{P}{P_E}\cdot\pi^2\frac{EI}{l}\cdot\frac{\delta}{l}$$
$$= Fl + \frac{P}{P_E}\pi^2 k\varnothing$$

Therefore
$$\varnothing = \frac{Fl}{2k\left[s(1+c) - \frac{\pi^2}{2}\cdot\frac{P}{P_E}\right]}$$

Also
$$M_{AB} = M_{BA} = -s(1+c)k\varnothing$$
$$= \frac{-Fl}{2\left[1 - \frac{\pi^2}{2s(1+c)}\cdot\frac{P}{P_E}\right]} = -m\frac{Fl}{2} \quad (1)$$

where

$$m = \frac{1}{1 - \frac{\pi^2}{2s\,(1+c)} \cdot \frac{P}{P_E}} \quad . \quad . \quad (2)$$

Both s and c are functions of P/P_E and therefore m is also a function of P/P_E and is shown in Fig. 3. m is a non-dimensional factor showing how much bigger the stanchion moments are than they would be if there were no $P\delta$ effects.

Also from equation (1)

$$\varnothing = \frac{mFl}{2s\,(1+c)k} \quad . \quad . \quad . \quad . \quad . \quad (3)$$

the sway angles are therefore not only increased due to the increased flexibility of the stanchions as P/P_E increases, but also they are multiplied by the same factor m.

Now consider equal rotations θ_B of joints B and B' without sway. The moments due to such rotations are given in the following pattern (in the manner due to Bolton[2])

Joint	A	B	C
Rotation of B	$(csk)_{AB}\theta_B$ $(sk)_{AB}\theta_B$	$4k_{BB'}\theta_B$	$(sk)_{BC}\theta_B$ $(csk)_{BC}\theta_B$
Rotation of B[1]		$2k_{BB'}\theta_B$	
Totals	$(csk)_{AB}\theta_B$ $(sk)_{AB}\theta_B$	$6k_{BB'}\theta_B$	$(sk)_{BC}\theta_B$ $(csk)_{BC}\theta_B$

As there is no sway there are no $P\delta$ terms and the shear in storey AB is given by

$$F = -\frac{(M_{AB} + M_{BA})}{l} = -\frac{\{s(1+c)k\}_{AB}\theta_B}{l} \quad (4)$$

If we allow storey AB to sway without rotation of joints to wipe out this shear, then from equation (3) the resulting sway angle will be given by

$$\varnothing_{AB} = \frac{m\,(-F)l}{2s\,(1+c)k}$$

Therefore from equation (4)

$$\varnothing_{AB} = \frac{m}{2}\,\theta_B \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

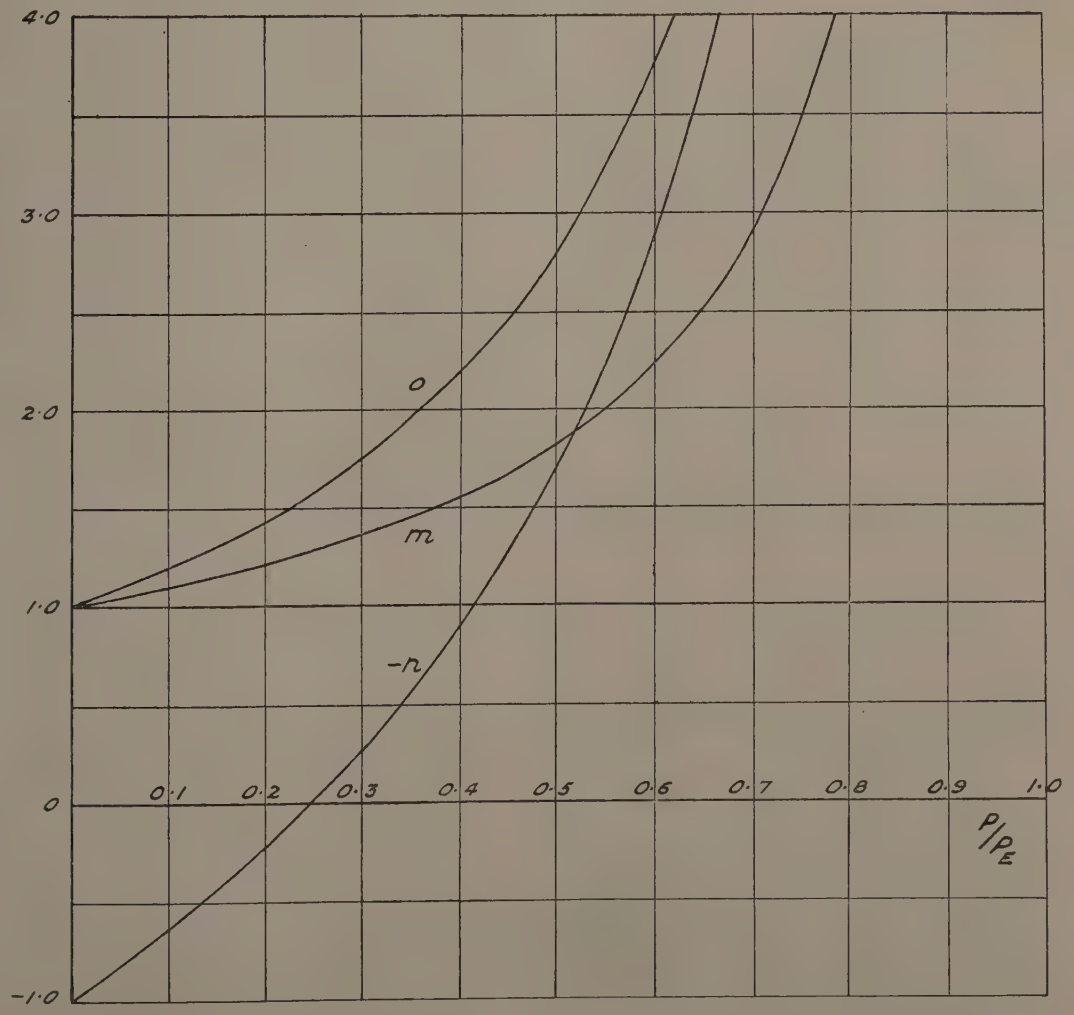


Fig. 3

Due to $\varnothing_{AB}$ the following moments will be caused :

$$M_{AB} = M_{BA} = -\left\{s(1+c)k\right\}_{AB}\varnothing_{AB} = -\left\{\frac{ms(1+c)k}{2}\right\}_{AB}\theta_B \quad . \quad . \quad (6)$$

Similarly, if we allow storey BC to sway without rotation of joints to wipe out the shear in storey BC the following moments will be caused :

$$M_{BC} = M_{CB} = -\left\{\frac{ms(1+c)k}{2}\right\}_{BC}\theta_B$$

The moment pattern due to equal rotations of B and B' combined with sways of storeys AB and BC so that no shears are set up in these storeys is therefore :

Joint	A	B	C
Moment	$\left\{sk\left[c-\frac{m(1+c)}{2}\right]\right\}_{AB}\theta_B$ $\left\{sk\left[1-\frac{m(1+c)}{2}\right]\right\}_{AB}\theta_B$	$6k_{BB'}\theta_B$	$\left\{sk\left[1-\frac{m(1+c)}{2}\right]\right\}_{BC}\theta_B$ $\left\{sk\left[c-\frac{m(1+c)}{2}\right]\right\}_{BC}\theta_B$

which we will write as :

A	B	C
$-(ok)_{AB}\theta_B$	$(nk)_{AB}\theta_B$ $6k_{BB'}\theta_B$ $(nk)_{BC}\theta_B$	$-(ok)_{BC}\theta_B$

. . . (7)

where

$$n = s\left[1-\frac{m(1+c)}{2}\right] \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8) \qquad o = s\left[-c+\frac{m(1+c)}{2}\right] \quad . \quad . \quad . \quad . \quad (9)$$

As s, c and m are all functions of P/P_E so are n and o. They are also shown in Fig. 3.

Now the particular property of a critical load is that at a critical load a small disturbance can take place without requiring any external forces. If we try to build up such a disturbance and use patterns of the type (7) we have already satisfied the condition that no shears are set up. In addition, there must be no external moment at each joint. This leads to as many equations as there are storeys, a typical equation being :

$$-(ok)_{AB}\theta_A + \left[(nk)_{AB} + 6k_{BB'} + (nk)_{BC}\right]\theta_B \; -(ok)_{BC}\theta_C = o \quad . \quad . \quad (10)$$

Suppose that we arrange for no external moments at any storey except the top one.

Then for a frame of N storeys we will have the set of equations :

$$\left.\begin{aligned}
&\left[(nk)_{01}+6k_{11'}+(nk)_{12}\right]\theta_1 - (ok)_{12}\theta_2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad = O \\
&-(ok)_{12}\theta_1 + \left[(nk)_{12}+6k_{22'}+(nk)_{23}\right]\theta_2 \; -(ok)_{23}\theta_3 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad = O \\
&\quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \\
&\quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \\
&\quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \\
&-(ok)_{N-2,\,N-1}\theta_{N-2} + \left[(nk)_{N-2,\,N-1}+6k_{N-1,\,(N-1)'}+(nk)_{N-1,\,N}\right]\theta_{N-1} - (ok)_{N-1,\,N}\theta_N \; = O \\
&-(ok)_{N-1,\,N}\theta_{N-1} + \left[(nk)_{N-1,\,N}+6k_{N,\,N'}\right]\theta_N = M
\end{aligned}\right\} \quad (11)$$

The set of equations (11) gives the relations between the rotations of the joints for two equal moments M applied at the topmost joints, there being no other disturbing forces applied to the frame. The equations give in succession the ratios $\frac{\theta_2}{\theta_1}, \frac{\theta_3}{\theta_1}$ and so on. The last equation determines θ_1 in terms of M and hence all the other θ values.

$$\text{Let } M = K\theta_N \text{ (defining } K) \quad . \quad . \quad . \quad . \quad (12)$$

For any particular value of the loading parameter the values of P/P_E for each stanchion length are known and hence the values of n and o. The coefficients in equation (11) are then known and the set can be solved to give the value of K. The lowest critical load is given by the lowest value of the loading parameter for which $K=O$ for then no moments will be required to maintain the deflected shape. The critical mode will however be determined ; the ratios of the θ's, but not their magnitude, being given by equations (11) and the associated $\varnothing$'s from equation (5).

Where the stanchion and beam stiffness vary the calculations will usually be done numerically, but we will illustrate the type of information given by this analysis by solving completely a simple family of structures.

Single Bay Frame with Constant Section Stanchions

Consider a symmetrical single bay frame in which the stanchions are of constant section throughout, all the storeys are the same height, the beams are of the same section and all the loads are carried at the topmost joints, as shown in Fig. 4. For this special simple case P/P_E is the same for each stanchion length and therefore all the values of n and o are equal. The stanchion and beam stiffnesses are also constant, say k_1 and k_2.

Equations (11) reduce to

$$\left.\begin{aligned} (2nk_1+6k_2)\theta_1 - ok_1\theta_2 &= O \\ -ok_1\theta_1+(2nk_1+6k_2)\theta_2 - ok_1\theta_3 &= O \\ \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot & \\ -ok_1\theta_{N-2}+(2nk_1+6k_2)\theta_{N-1} - ok_1\theta_N &= O \\ -ok_1\theta_{N-1}+(nk_1+6k_1)\theta_N &= M \end{aligned}\right\} (13)$$

The next to the last equation may be written as

$$\theta_{N-2} - X\theta_{N-1} + \theta_N = O \quad . \quad . \quad . \quad . \quad . \quad (14)$$

where

$$X = \frac{\left(2n + 6\frac{k_2}{k_1}\right)}{o} \quad . \quad . \quad . \quad (15)$$

Equation (14) in fact contains all the equations of the set except the last one. It is a difference equation which is satisfied by $\theta_N = x^N$ if

$$x^{N-2} - Xx^{N-1} + x^N = O$$

$$\text{i.e.} \quad 1 - Xx + x^2 = O \quad . \quad . \quad . \quad . \quad (16)$$

Let λ, μ be the roots of equation (16) i.e.

$$\begin{matrix}\lambda \\ \mu\end{matrix} = \frac{1}{2}\left[X \pm \sqrt{X^2-4}\right]$$

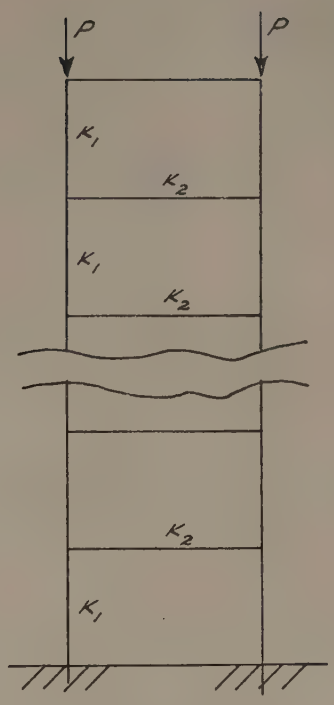


Fig. 4

Then $$\theta_N = C_1\lambda^N + C_2\mu^N$$

is the complete solution of equation (14).

Now $\theta_o = O \quad \theta_1 = \theta_1$

Therefore $$O = C_1 + C_2$$

$$\theta_1 = C_1\lambda + C_2\mu = C_1(\lambda - \mu)$$

and therefore $$\theta_N = \theta_1 \cdot \frac{\lambda^N - \mu^N}{\lambda - \mu} \quad . \quad . \quad (17)$$

is the solution unless $$\lambda = \mu$$

If $\lambda = \mu$ then $X = 2$ and equation (14) becomes

$$(\theta_N - \theta_{N-1}) = (\theta_{N-1} - \theta_{N-2})$$

giving $$\theta_N = N\theta_1 \quad . \quad . \quad (18)$$

The critical load is given by equating M to zero in the last of equations (13) i.e. by

$$\theta_{N-1} + \left(-X + \frac{n}{o}\right)\theta_N = O \quad . \quad . \quad . \quad (19)$$

Substituting from equation (17) we obtain

$$X - \frac{n}{o} = \frac{\lambda^{N-1} - \mu^{N-1}}{\lambda^N - \mu^N} \quad . \quad . \quad . \quad . \quad . \quad (20)$$

Equation (20) is a general equation giving the critical load for a frame of N storeys ($\lambda \neq \mu$).

We have in succession

$$N = 1 \qquad X - \frac{n}{o} = 0 \quad . \quad (21)$$

$$N = 2 \qquad X\left(X - \frac{n}{o}\right) = 1 \quad . \quad (22)$$

$$N = 3 \qquad \left(X^2 - 1\right)\left(X - \frac{n}{o}\right) = X \quad . \quad (23)$$

$$N = 4 \qquad X\left(X^2 - 2\right)\left(X - \frac{n}{n}\right) = X^2 - 1 \quad (24)$$

and so on.

Equations (21)—(24) can be easily solved. For any particular value of X they give the corresponding value of n/o and therefore of n and o separately.

$\frac{k_2}{k_1}$ is then obtained from equation (15).

For no case does $X = 2$ and therefore $\lambda \neq \mu$ and the solutions can be accepted. The resulting $\frac{P_C}{P_E}$, $\frac{k_2}{k_1}$ curves are shown in Fig. 5; also shown is a curve for an infinite number of storeys which is obtained as follows:

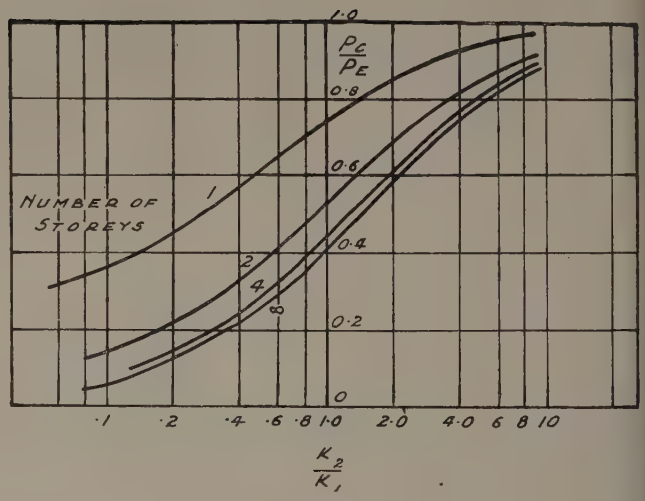


Fig. 5

For all the cases solved $X < 2$ and therefore λ and μ are imaginary, also $\lambda\mu = +1$. We may therefore write

$$\lambda = e^{i\omega} \quad \text{and} \quad \mu = e^{-i\omega}$$

where

$$\tan\omega = \frac{\sqrt{4 - X^2}}{X} \quad \text{and} \quad \cos\omega = \frac{X}{2} \qquad (25)$$

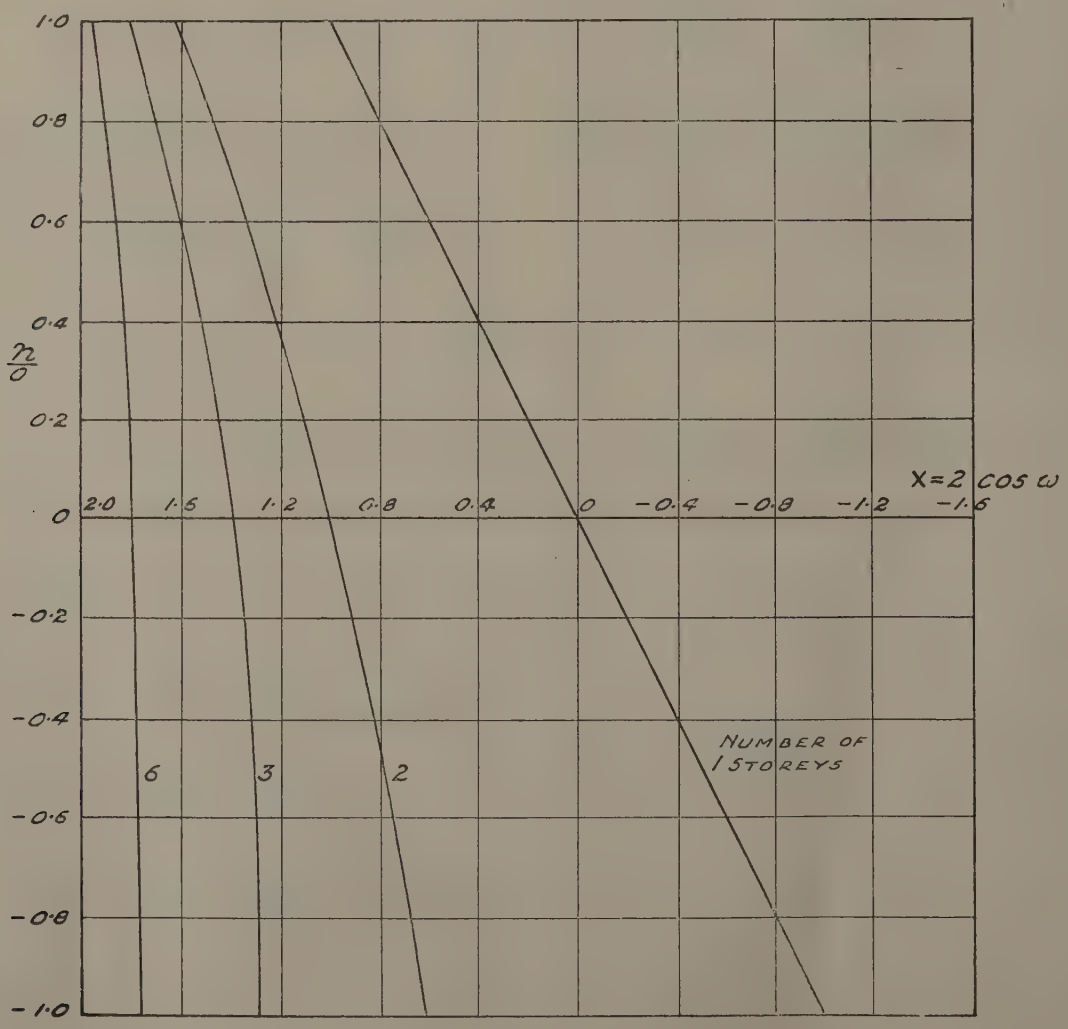


Fig. 6

Equation (20) becomes

$$2\cos\omega - \frac{n}{o} = \frac{e^{(N-1)i\omega} - e^{-(N-1)i\omega}}{e^{Ni\omega} - e^{-Ni\omega}}$$

$$= \frac{\sin(N-1)\omega}{\sin N\omega}$$

$$= \cos\omega - \frac{\sin\omega}{\tan N\omega}$$

$$\text{i.e.} \qquad \frac{n}{o} = \cos\omega + \frac{\sin\omega}{\tan N\omega}. \quad . \quad . \quad . \quad . \quad (26)$$

$$\omega \neq n\pi$$

Equation (26) is another form of equation (20).

To obtain the lowest critical load for a given value of $\frac{P}{P_E}$, $\frac{k_2}{k_1}$ must be as big as possible (see Fig. 5).

Now $X = +\frac{\left(2n + 6\frac{k_2}{k_1}\right)}{o}$ and therefore X must also be as big as possible. Therefore from equation (25) ω must be as small as possible and we have to take the lowest root of equation (26). As N increases indefinitely we see from Fig. 6 that the limiting solution is given by $\omega = O$ and therefore from equation (25) by $X = 2$ which enables the curve for the infinite number of storeys to be plotted on Fig. 5.

Many Bay Frames

The solution we have obtained for a single bay frame also serves as the solution for a frame with an infinite number of bays if we treat k_2 as the sum of the stiffnesses of the beams on the two sides of a stanchion, i.e. as twice the stiffness of a single beam. This is obvious from Fig. 7. For frames with a finite number of bays the critical load will have a value intermediate between those of the two extreme cases.

Historical Note

Karman and Biot[3] have discussed the critical loads of a frame similar to Fig. 4, but with the top member prevented from translating and the top joints from rotating. This problem is of less direct interest to structural engineers, but for the limiting case of a large number of storeys their solution is in agreement with ours.

Bleich has discussed the large number of bays case[4] but not found the limiting solution for a large number of storeys. He also gives a solution for the particular case of a single bay two storey frame.[5] It has been pointed out to us by our research student, Mr. D. B. Chandler, that this last solution is in error giving for example

$$\frac{P_C}{P_E} = 0.594 \text{ instead of } \frac{P_C}{P_E} = 0.524 \text{ at } \frac{k_2}{k_1} = 1.0$$

In any case, the single bay solution is not compatible with the large number of bays case and the connection between them does not seem to have been realised.

The first draft of this paper was prepared in November, 1952 before seeing Bleich's book. It has seemed worthwhile to present this paper not only because of the points made above, but also because of the greater simplicity of the methods contained herein.

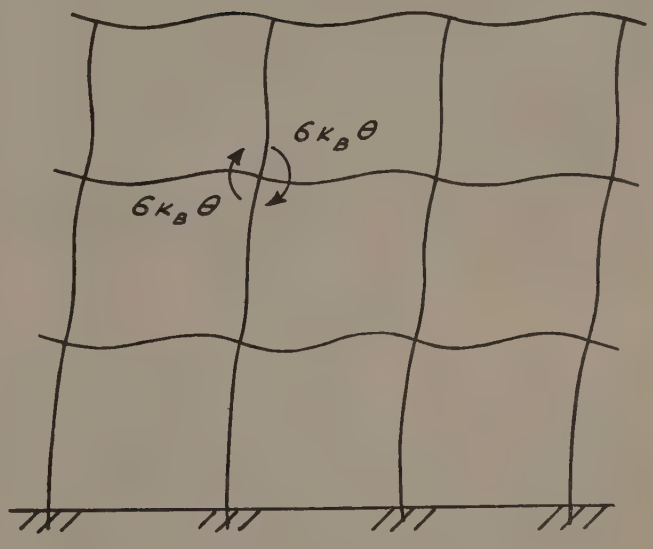


Fig. 7

Acknowledgment

This paper forms part of a research project, on the effect of stability on the failure loads of structures, being undertaken in the Department of Building and Structural Engineering, College of Technology, Manchester.

References

[1] W. Merchant, "The Failure Load of Rigid Jointed Frameworks as Influenced by Stability," *Journal Inst. Struct. Eng.*, July, 1954.
[2] A. Bolton, "A New Approach to the Elastic Analysis of Two Dimensional Rigid Frames," *Journal Inst. Struct. Eng.*, January, 1952.
[3] Karman & Biot, "Mathematical Methods in Engineering," *McGraw Hill Book Co.*, 1940, chap. xi.
[4] Bleich, "Buckling Strength of Metal Structures," *McGraw Hill Book Co.*, 1952, chap. vii, p. 260.
[5] Bleich, *Ibid.*, chap. vii, p. 255.

Since the preparation of this paper a useful series of tables of stability functions has been prepared on the Manchester University Electronic Computer. They are "Tables of Stiffness and Carry-over Factors for Structural Members under Axial Load" by R. K. Livesley to whom application should be made for copies.

The tables include not only the usual s and c functions, but also the m, n, o functions as introduced in this paper.

A Quick Approximation to the Critical Load of Rigidly Jointed Trusses

By Arthur Bolton, M.Sc.Tech. (Graduate)

Need for the Calculation of Critical Loads

Engineers are rightly cautious in reviewing new techniques of structural design and indeed are sometimes regarded as being too conservative in their acceptance of the results of engineering research. It seems to the writer however that any further progress in the science of steelwork design can be only made on the basis of new assumptions.

Existing methods of design guard against the local collapse of individual parts of the structure but lead to such great variability in load factor of the whole that even a small increase in " allowable stresses " might result in an apparently safe design being dangerously near collapse. However, another design on the same " allowable stresses " might be proved by a loading test to be over-generous.

In a new approach which has been suggested, it is necessary to find the critical load for a complete truss or portal frame. This is not too difficult when techniques based on moment distribution are used rather than a formal mathematical solution.

Introduction

The design methods used for steel structures have been built up over a long period of time. In the beginning, assumptions were made which were reasonable considering the existing state of knowledge and constructional ability. On these assumptions modern engineering has built up a mathematical superstructure and refined it to an astonishing extent, but the basic assumptions seem to be unquestioned whenever a new examination is made of design methods. Modern design thus suffers from an attempt to obtain a solution within a small percentage error when the basic theory is inaccurate to a far larger degree.

Some of the fundamental difficulties are these :

1. An elastic theory does not represent the behaviour of the structure, and is therefore incapable of being refined to give the utmost economy of material without the risk of failures taking place.

Elastic theory is applicable for the rapidly fluctuating stresses of mechanical engineering, but for slowly applied loads plastic action occurs in many parts of the structure even at the working load. For example, in a riveted or bolted joint elastic theory shows that even with perfect manufacture certain rivets or bolts would reach the yield point whilst the average for the whole joint was far below yield.[1]

2. It is pointless to refer to a " safe stress " for a steel member.

If the stress in the member were a real criterion of failure internal stresses for the unloaded member would need to be known since in an ordinary rolled section for instance, stresses due to unequal cooling and straightening may amount to about three quarters of the yield stress.[2]

Moreover, stress concentration factors at holes, changes of section etc. would need to be considered.

3. A factor of safety based on stresses has little connection with the safety of the structure. A member close to its failure load may have a large factor of safety. Alternatively a structure far from failure may have a small factor of safety.

4. The theory used in existing methods can only be applied with certainty to isolated members, because the empirical values, for instance in the Perry-Robertson formula, have been obtained by experiments on isolated members.

This is perhaps the greatest shortcoming in existing design. It can easily be shown that a continuous beam, designed in exactly the same way, for the same stresses and factor of safety as a simply supported beam, nevertheless has a greater load carrying capacity because of its redundancies.

In addition because of its shape an I section used as a gantry girder say, has a far bigger reserve of strength against lateral forces than it has against vertical loads.

Even more seriously in error is the position regarding stanchion design because the behaviour is more complex.

Obviously the semi-rigid method can only give the best answer by a lucky chance, since parts of the structure are assumed to be acting differently in different parts of the calculation.

If a more elaborate calculation by Moment Distribution or Slope Deflection is made, it gives an unreal bending moment diagram, because the stiffness of a stanchion carrying its working load is much less than the elastic stiffness used in the calculation. This introduces a vicious spiral of errors[3]; the exaggerated stanchion stiffness attracts a bigger bending moment in the calculation (but not in the real structure) and so a bigger stanchion is used, which in turn attracts an even bigger moment.

Even if proper allowance is made for the reduced stanchion stiffnesses and an elastic calculation carried out by moment distribution, or if a plastic design method is used, there is no justification for taking a single length of a continuous stanchion and designing it from tables using :

(*a*) The bending moments at the design load multiplied by a constant ;

(*b*) The axial load in the stanchion multiplied by the same constant ;

(*c*) An arbitrary effective length.

These three quantities are not independent. A stanchion carrying a particular value of end moment applied by beams at a certain load will not carry double the moment at double the load. It may carry only a fraction of the moment at the greater load. Alternatively it may carry a greater moment of the opposite sign.

Moreover, even if the effective length of the stanchion was known at the design load it would not be the same at the greater load, because it is continuously reduced as the axial load is increased.

To summarise, the theory used for design does not represent the behaviour of the structure, nor do the empirical constants given in design codes. They result in a structure which is sufficient in conventional cases to carry the loads, but which may be so generously designed as to carry many times the actual load without failure ; nor is any indication given of the members which are most inefficiently used. There is also the possibility to be considered that as structures change their present architectural form the empirical values may give a structure which is insufficient.

The divergence between " theory " and the real structure has led in the recent past to an inaccurate theory actually producing weaker structures. When using a ductile material like mild steel for steady loads rigidity of connections can only strengthen the structure. However, the introduction of pinned joints and such devices as turning a member so that it bends about its weaker axis, does considerably weaken the structure in spite of there being an apparent improvement in " allowable values."

A method of avoiding these difficulties has been suggested by W. Merchant.[4] Just as the Perry-Robertson Formula is a function of the critical load at which the strut becomes unstable, and the limit load at which the whole section reaches the yield stress, so Merchant has suggested that the failure load of a complete structure can be regarded as a function of the critical load of the structure as a whole, and the limit load at which there are enough plastic hinges in the structure to allow collapse.

Sufficient work has been carried out and published so that the limit load of a steel frame can be obtained reasonably quickly.[5] It is for critical load determination that simpler methods need to be developed for practical design.

This paper suggests a method which will give a quick approximation to the critical load of trusses in which the joints are not allowed to displace, or for building frames in which sway is prevented by bracing or by the use of rigid panel filling.

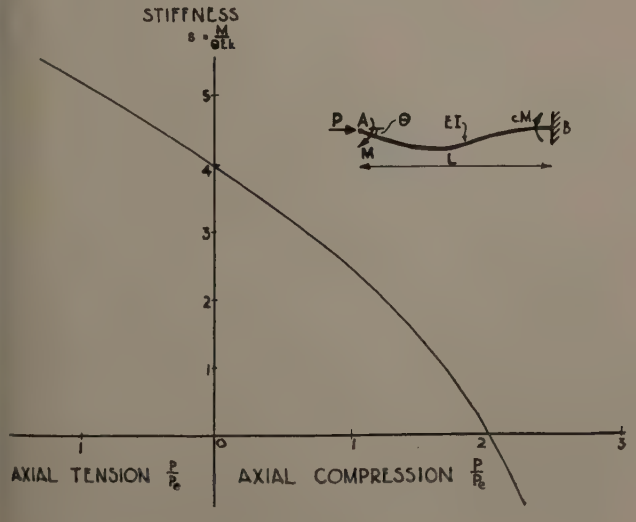


Fig. 1

Stiffness and Carry-over Factor

If a uniform member is subjected to an axial load and then the stiffness of one end against rotation (when the other end is fixed) is determined, the relation between stiffness and the axial load is shown in Fig. 1.

The stiffness is obtained by noting the rotation θ for a given testing moment applied. At zero axial load, i.e. in the case of a beam

$$M = 4EI/L \times \theta = 4Ek\theta$$

Then the stiffness $= M/\theta = 4Ek$.

It is convenient to consider a non-dimensional form of stiffness s in the general case where :

$$M = s\,(Ek\theta)$$

It will be seen from Fig. 1 that the stiffness decreases as the axial compression is increased, until a point is reached at which s is zero. At this point there is no resistance to a moment applied at A and the member becomes unstable.

If P is increased beyond the value corresponding to zero stiffness, it will be found necessary to restrain end A to maintain the equilibrium of AB. The stiffness of A against rotation is now negative and AB will reduce the stiffness of any other member connected to A. If AB is subjected to a tensile force the stiffness of A against rotation is increased.

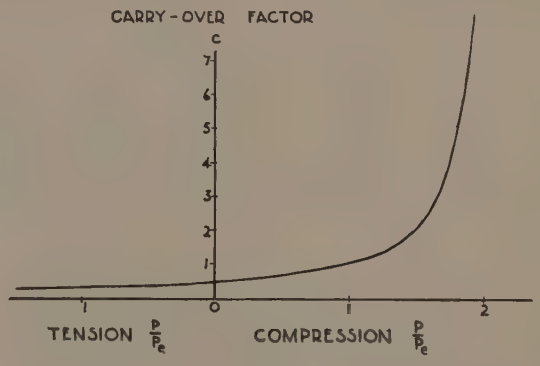


Fig. 2

When a test is made of the moment carried over to B when A is rotated, the relationship between the carry-over factor c and the axial load is as shown in Fig. 2. At zero axial load i.e. for a beam, the carry-over factor is a half, and this factor increases continuously as the compression in AB increases. For example at the Euler Load the carry-over factor is unity and above this load the carry-over factor rises sharply. When AB is in tension the value of the carry-over factor is reduced, for example at a tensile load equal to the Euler Load the carry-over factor is only about one third.

Critical Loads in Trusses

It will be obvious from Fig. 1 that if a number of members meet at a joint A, one of them being a strut, and the axial loads in all the members are increased in the same proportion the total stiffness of the joint will eventually be reduced. This is because the negative

gradient of the stiffness curve increases as the compressive load increases and therefore the reduction in stiffness of the strut for increments of load will be greater than the increase in stiffness of the ties.

Thus if the loads continue to increase, the stiffness of joint A will eventually become zero. At this load the joint offers no resistance to rotation and the truss becomes unstable. There is a difference between the case of the truss and that of the test on a single member described above. In the case of the truss the joints adjacent to A will rotate an amount depending on their stiffnesses when A is rotated, and to this extent the stiffness of joint A depends upon the stiffness of all other joints of the truss.

Methods have been described which obtain the critical loads of trusses by obtaining the stiffness curve of a typical joint.[6] A load factor is chosen, the forces in the members of the truss obtained and hence the values of s and c for each member found from tables. An arbitrary moment of say 100 units is applied at the joint A being considered and the moments carried over to adjacent joints are obtained. These other joints are balanced in turn, as in normal Moment Distribution, until every joint in the truss except A is balanced. The moments in the members meeting at A are then added and will be found to be considerably less than 100. Dividing this moment by the original rotation of A will give a point on the stiffness curve of joint A.

A greater load is now chosen and the process repeated. A curve roughly similar in shape to that of Fig. 1 will be produced, and eventually the load at which the stiffness of A is zero can be obtained by drawing a curve through the points plotted from the above determinations.

The objection to these methods is that several complete moment distribution or relaxation type calculations have to be made for the determination of the critical load. Worse than this, as the stiffness of the truss approaches zero, the convergence of the calculation becomes very slow and many cycles are required. As many as twenty-five cycles have been found necessary in quite simple trusses.

Basis of the Approximation

The approximation demonstrated in this paper differs from the general " moment distribution " kind of solution in two respects. Firstly the joint to be rotated is chosen in a way which best fits the calculation and secondly the calculation is reduced to one cycle only as in the method of remove frames[7] or other degree of fixity methods. This also has the effect of reducing the number of members to be considered in a large frame.

It has been said[8] that the critical load of a truss can be determined by testing any joint in the way described above. This is not strictly true because if the joint tested has a zero rotation in the lowest critical mode (as might happen with symmetrical structures say) the critical load indicated by this method will usually not be the lowest.

Testing an arbitrarily chosen joint may give a stiffness curve of sharp curvature near the critical load which needs a larger number of sets of calculation to obtain its shape. Moreover, each one of these sets of calculation will be found to be less convergent in this case than is necessary.

These difficulties can be avoided and accuracy increased, if the most suitable joint is tested. This joint can be easily determined. The value of P/P_e for every member is calculated, where :

P is the axial load carried by the member (compression positive) ;

P_e is the Euler load for the member $\left(\frac{\pi^2 EI}{L^2}\right)$

Then the critical joint in general will be one of the two joints at the end of the strut with the biggest value of P/P_e, and of the two it will be the one with the lower total stiffness. This joint is the one which rotates farthest when the truss carries a load nearly equal to the critical load. Rotation of this joint distorts the truss into a mode closer to the lowest critical mode than the rotation of any other joint. The stiffness of this particular joint falls off more quickly than any other since it includes a member on the steepest parts of the s and c curves.

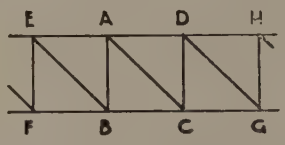


Fig. 3

It has been shown that stronger members at a joint support the weakest as it stiffness becomes zero, and that likewise the weakest joint is supported by adjacent joints until the whole strength of the truss has been used. But although adjacent joints can contribute appreciably to the strength of the weakest, those more indirectly connected have very little effect. For instance in Fig. 3 if A is the weakest joint, B, E, D and C can stiffen A, but joints like F, G and H will have a very much smaller effect.

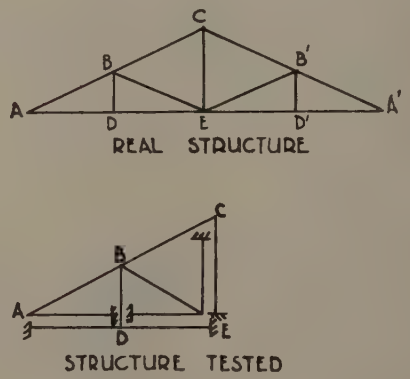


Fig. 4

The method then is to consider a simplified truss in which the rotation of joints not directly connected to the one under review is ignored. In this case there will be no need for repeated cycles and the time required for a determination of the critical load is greatly reduced. This method of reducing the size of the frame has been found to give reasonable results for elastic frames. The comparison between the real structure and the structure tested is shown in Fig. 4.

	A		B	
DISTRIBUTION COEFFICIENTS	·8	·2	·7	·3
DISTRIBUTE 100 UNITS AT A		20		
CARRY-OVER TO B			10	
BALANCE B			-7	
CARRY-OVER TO A		-3·5		
TOTAL MOMENT AT AB		16·5		

Fig. 5

Method

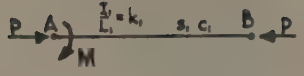
Fig. 5 shows part of a moment distribution solution for one span AB of a continuous beam when 100 units of bending moment are balanced at A. Fig. 6 shows exactly the same procedure for a stability problem when the member carries axial loads and hence has different stiffness and carry-over factors.

The first step in each case is to find the distribution coefficients.

In a practical problem the members will be of different lengths and moments of inertia; as well as having an s value depending on the axial load carried. If there were no axial load in a member its stiffness would be proportional to k where $k = I/L$. When known axial loads are carried the stiffness will be $k \times s$ where s is obtained from a graph such as Fig. 1 or from tables.[11, 10]

From these stiffness values the distribution factors

T_A DENOTES TOTAL STIFFNESS AT 'A'.



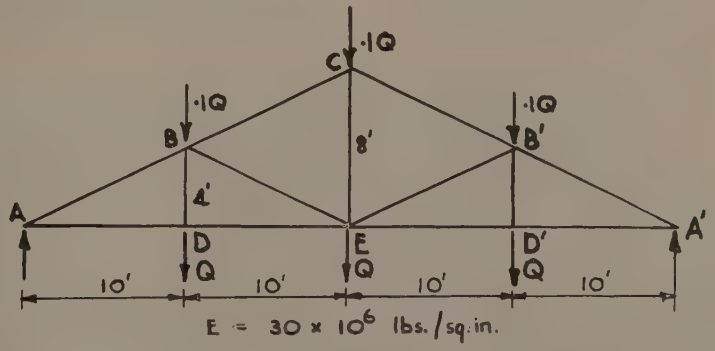
DISTRIBUTION COEFFICIENTS	$\frac{k_1 s_1}{T_A}$	$\frac{k_1 s_1}{T_B}$
DISTRIBUTE M AT 'A'.	$M \cdot \frac{k_1 s_1}{T_A}$	
CARRY-OVER TO 'B'		$c_1 M \frac{k_1 s_1}{T_A}$
BALANCE 'B'		$-\frac{k_1 s_1}{T_B} c_1 M \frac{k_1 s_1}{T_A}$
CARRY-OVER TO 'A'	$-c_1 \cdot \frac{k_1 s_1}{T_B} \cdot c_1 \cdot M \cdot \frac{k_1 s_1}{T_A}$	
TOTAL MOMENT AT AB	$M\left[\frac{k_1 s_1}{T_A} - \frac{(k_1 s_1 c_1)^2}{T_A \cdot T_B}\right]$	

Fig. 6

can be worked out as for normal Moment Distribution. Now if a moment M is applied at A the moment carried by AB will be $M \times$ the distribution coefficient for AB, and the moment carried over to B will be c times this amount. There will now exist an out of balance moment at B which can be balanced as usual. The carry-over to A from the share of balancing moment appearing at BA can be obtained and so the total moment appearing at AB can be found as

$$M\left[\frac{k_1 s_1}{T_A} - \frac{(k_1 s_1 c_1)^2}{T_A \cdot T_B}\right]$$

where T_A represents the total stiffness $\Sigma(k \times s)$ of members meeting at A. All these operations are tabulated in Fig. 6.



MEMBER	SECTION	$k = \frac{I}{L}$ RATIO	Q = 38,000 lbs. $\frac{P}{P_e}$	ks	$\frac{(ksc)^2}{T_{FAR}}$	Q = 39,000 lbs. $\frac{P}{P_e}$	ks	$\frac{(ksc)^2}{T_{FAR}}$	Q = 40,000 lbs. $\frac{P}{P_e}$	ks	$\frac{(ksc)^2}{T_{FAR}}$
AB	2/ 3"×3"×⅝" L	1	1·83	0·63	1·77	1·88	0·50	1·89	1·93	0·35	1·98
BC	2/ 3"×3"×⅝" L	1	1·20	2·08	2·35	1·23	2·03	2·41	1·26	1·96	2·49
AD DE	2/ 3"×3"×½" L	0·92	-1·73	5·42		-1·78	5·46		-1·82	5·54	
BD	2/ 2"×2"×¼" L	0·36	-0·42	1·63	0·04	-0·43	1·64	0·04	-0·44	1·64	0·04
BE	2/ 2½"×2½"×⅜" L	0·38	1·59	0·47	0·18	1·63	0·44	0·20	1·67	0·40	0·20
CE	2/ 2"×2"×⅜" L	0·25	-2·57	1·67		-2·64	1·68		-2·80	1·70	
		T_A	6·05	TOTAL	4·34	5·96		4·54	5·89		4·71
		T_B	4·81			4·61			4·15		
		½T_C	2·91	$\frac{4·34}{4·81} = 0·90$		2·87	$\frac{4·54}{4·61} = 0·98$		2·81	$\frac{4·71}{4·15} = 1·13$	
		T_D	12·47	STABLE		12·56	BARELY STABLE		12·72	UNSTABLE	
		½T_E	6·72			6·74			6·79		

Fig. 7

This is the result for a rotation of A on one of the members meeting at A. Obviously the total moment caused at A will be the sum of such terms for all members meeting at A, i.e.

$$M\left[1 - \Sigma\frac{(k_1 s_1 c_1)^2}{T_A \cdot T_B}\right]$$

If this quantity is positive, rotation of A causes a moment to appear at A and work will be required to rotate A. If the term is zero, no moment will appear at A when it is rotated, and therefore no energy is required to rotate A. This is an indication that joint A is on the verge of instability. If the term is negative it indicates that if A is rotated a moment of opposite sign to the displacement is required to hold A in equilibrium and therefore the truss has passed its critical load.

The truss is first analysed graphically or by method of sections and the axial load in every member for a unit load pattern obtained. The Euler load for each member and the k values are also calculated.

A certain multiple of the load pattern is now chosen, and the value of the axial load, as a proportion of the Euler load for each member is calculated. From tables the value of s and c are obtained and so the strut nearest to failure can be chosen. The total stiffness T of the members meeting at each joint is calculated and the critical joint chosen.

For this joint, $(k.s.c)^2$ divided by the total stiffness of the joint remote from the critical one, is tabulated for each member. Dividing the sum of these terms by the stiffness of the critical joint yields the criterion which indicates whether the load being investigated is the critical load or not.

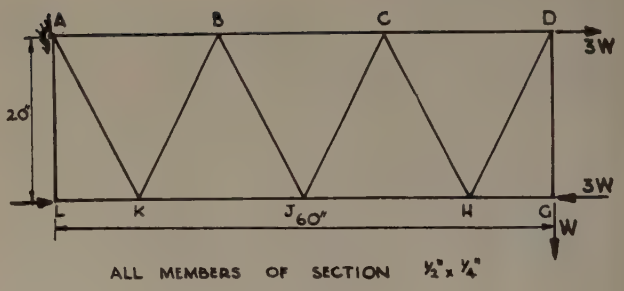


Fig. 8

Numerical Examples

In reference 9 a roof truss was investigated by 'exact' methods. The critical load was shown to lie between $Q = 38{,}000$ lbs. and $Q = 39{,}000$ lbs. (Actually it is almost 39,000 lbs.) This truss has been investigated by the method described here and the calculation is set out in Fig. 7. On checking B is found to be the critical joint. This results in a feature which is not general. In this case the structure and the load are both symmetrical about a vertical centre line. Hence both B and B' become primarily unstable at the same load, which means that C and E would each receive carry-over moments from both B and B'. To allow for this doubling with as little alteration in the table as possible, only half the total stiffness is reckoned at C and E. This really means that only half of the total stiffness of member CE can be credited to each of the joints B and B'. The calculation gives by interpolation the critical load for the truss as $Q = 39{,}200$ lbs.

MEMBER	$\frac{l}{L} = k$ RATIO	P_e lbs.	WITH SHORT GUSSETS W = 87·5 lbs. $\frac{P}{P_e}$	k.s.	$\frac{(ksc)^2}{T_{FAR}}$	W = 90 lbs. $\frac{P}{P_e}$	k.s.	$\frac{(ksc)^2}{T_{FAR}}$	WITH LONG GUSSETS W = 100 lbs. k.s.	c	$\frac{(ksc)^2}{T_{FAR}}$	W = 105 lbs. k.s.	c	$\frac{(ksc)^2}{T_{FAR}}$
AB	1	168·5	-2·86	6·80		-2·94	6·97		9·63			9·82		
BC	1	168·5	-2·34	6·41		-2·41	6·50		8·92			9·08		
CD	1	168·5	-1·83	6·00		-1·87	6·01		8·15			8·30		
LK	2	674·0	0·78	5·70		0·80	5·78		3·40			3·10		
KJ	1	168·5	2·60	-2·28	2·40	2·68	-2·72	2·69	-2·60	-1·91	1·88	-3·48	-1·67	2·93
JH	1	168·5	2·08	-0·11	0·98	2·14	-0·11	1·02	-0·30	-10·0	0·54	-0·80	-4·76	0·90
HG	2	674·0	0·39	6·92		0·40	6·86		4·10			4·00		
AL	1	168·5	0	4		0	4		5·08			5·08		
AK BJ CH	·895	135·0	-0·73	4·36	0·12	-0·74	4·38	0·13	6·38	0·43	0·21	6·44	0·42	0·21
KB JC HD	·895	135·0	0·73	2·60	0·22	0·74	2·60	0·23	3·48	0·89	0·29	3·40	0·92	0·30
DG	1	168·5	-0·52	4·63		-0·53	4·65		6·02			5·70		
					3·72			4·07			2·92			4·34
	T_B		20·17			20·27			27·4			27·7		
	T_C		19·37			19·49			25·9			26·2		
	T_H		13·77	$\frac{3·72}{4·57} = 0·82$		13·73	$\frac{4·07}{4·15} = 0·98$		16·7	$\frac{2·92}{5·91} = 0·50$		16·0	$\frac{4·34}{4·52} = 0·96$	
	T_J		4·57			4·15			5·91			4·52		
	T_K		10·39			10·04			13·01			11·52		

Fig. 9

Fig. 8 shows a truss checked analytically and experimentally in reference 8 pp. 24-29. In this paper two sets of calculations were made, firstly as though the members were rigidly connected together but with gussets so short that the ends of the members were not reinforced (here referred to as " with short gussets "), and secondly with practical sized gussets of which the stiffening effect was calculated (here referred to as " with long gussets ").

The calculations using the approximation are set out in Fig. 9. To simplify comparison only two values of the load are checked in each case. For the calculation including the effect of gussets, the stiffness and carry-over values given in appendix 2 reference 8 are used.

When the values of $k.s$ have been obtained for each member, J is seen to be the weakest joint and the approximate critical load is obtained by investigating joint J and the adjacent joints B, C, H and K only. This calculation is shown in Fig. 9.

Linear extrapolation gives the approximate critical loads with short gussets as $W = 90.3$ lbs. against $W = 87.5$ lbs. in the reference, and with long gussets as $W = 105.3$ lbs. against $W = 102.5 \pm 2.5$ lbs. The errors are thus $+3.2$ per cent. and $+2.7$ per cent. of the more elaborately calculated values. The writer believes that it is unnecessary to determine critical loads to a closer tolerance than say ± 5 per cent. in view of other tolerances of at least the same order existing in the physical properties of the structure and the loads it carried.

However, if a closer approximation is required it can be obtained, at the expense of more labour, by correcting the stiffness of adjacent joints, again using degree of fixity methods.

In reference 11 values will be found tabulated which are very convenient for the calculations given in this paper.

This paper continues a research project on the effect of stability on the failure loads of structures being undertaken in the Department of Building and Structural Engineering, College of Technology, Manchester.

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[7] E. E. Lundquist, " Stability of Structural Members under Axial Loads," *Technical Note No.* 617, *N.A.C.A.*, 1937.

[8] R. H. Wood, " An Economical Design of Rigid Steel Frames for Multi-Storey Buildings," *H.M.S.O. National Building Studies Research Paper No.* 10.

[9] N. S. Hoff, B. A. Boley, S. V. Nardo, and S. Kaufman, " Buckling of Rigid Jointed Plane Trusses," *Proc. A.S.C.E.*, Vol. 76, June 1950.

[10] C. Winter, P. T. Hsu, S. Koo, and M. H. Loh, " Buckling of Trusses and Rigid Frames," *Bulletin No.* 36, *Cornell Univ. Engineering Experimental Station, Ithaca*, April 1948.

[11] E. E. Lundquist and W. D. Kroll, " Extended Tables of Stiffness and Carry-over Factor for Structural Members under Axial Load," *N.A.C.A., A.R.R. No.* 4B24.

[12] R. K. Livesley, " Tables of Stiffness and Carry-over Factors for Structural Members under Axial Load, *Manchester Univ. Computing Machine Laboratory*.

Book Reviews

(*continued*)

Holzbau-Taschenbuch, edited by R. von Halasz. (Berlin : Wm. Ernst & Son, 1952.) 4th Edition. 427 pp., 510 Figs. $6\frac{1}{2}$ in. $\times$ $4\frac{3}{4}$ in. Price DM. 19.50.

This is the fourth edition of a useful and comprehensive handbook on timber and timber structures It has been completely revised and brought up-to-date and new chapters have been added.

Chapters included in the handbook deal with : Timber and timber products, sources, properties, preservation, standards, dimensions and fundamental structural principles ; Timber frameworks ; Wood joints ; Nailed structures ; Glued structures ; Ceilings in dwelling-houses ; Timber roof supporting structures ; Halls, bridges, houses and barracks constructed in timber ; Scaffolding ; Timber in river and harbour work ; and Tools and Machines for timber work.

The volume also contains useful tables, specifications and a comprehensive bibliography.

Baustoffverarbeitung und Baustellenprüfung des Betons, 2nd Edition, by A. Kleinlogel. (Berlin : Walter Gruyter, 1951.) $6\frac{1}{10}$ in. $\times$ 4 in. 126 pp., 35 Figs. Price DM. 2.40.

This is a completely revised and expanded edition of a useful practical booklet on the production and testing of concrete. The work is divided into two parts, the first dealing with the physical properties, composition, etc. of the materials used in the manufacture of concrete—cements, aggregates, admixtures, water and steel reinforcement. The second part deals with the manufacture of various kinds of concrete, including the mix proportions, methods of mixing, storage and durability, influence of weather, including concreting in winter.

Prestressed Concrete, 3rd Edition, by G. Magnel. (London : Concrete Publications, 1954.) 345 + vi pp. 9 in. $\times$ 6 in. Price 20s.

The general arrangement of the third edition of this well-known book has not been changed, but many chapters have been extended and others added. The new material includes a table of dimensions of beams for various spans and loads, together with the number of cables required, and a method of calculating the ultimate strength of beams based on tests made on full-size beams. Data are given on economical spans for continuous beams and on their breaking strength, and details of a number of tests are included. The chapter on the applications of prestressed concrete includes many new bridges, building frames, and a flat-slab floor to carry very heavy loads. The examples are taken from all parts of the world.

Foundations, Underpinning and Structural Problems at *The Daily News* Building in the City of London*

Discussion on paper by Mr. Frederick W. Slatter, M.I.Struct.E., M.Cons.E., M.Inst.W., M.Soc.C.E.(France), and Mr. Arthur Brown, A.M.I.Mech.E., M.Soc.C.E.(France)

THE PRESIDENT proposed a vote of thanks to the Authors for the skill with which they had put together the information presented in the paper and the clarity of the photographs and drawings and descriptions they had given.

(The vote of thinks was heartily accorded.)

Mr. Kenneth SEVERN, M.C. (Associate-Member of Council) said a great deal of thought was required in tackling foundation problems on sites restricted by neighbouring premises, particularly where the new construction was carried below the levels of adjoining footings. The Authors had shown no lack of ingenuity in dealing with their many problems, and that was the more praiseworthy when one remembered the enormous amount of administration involved in any job of that kind. Apart from the normal liaison with client, architect and contractor, the owners of adjoining buildings must be kept constantly informed of any change of plan, and their own consultants given an opportunity to consider new designs and revised methods of construction.

Mr. Severn was particularly interested in the two methods used to form the permanent shuttering against earth faces—an in situ concrete skin in the case of underpinning, and the precast concrete poling boards at the back of retaining walls ; and he asked for the Authors' further views on the relative advantages of those methods. At first sight they would appear to have reversed the order. In underpinning, where placing concrete was awkward and effective vibration virtually impossible, a precast poling board would seem to be the better proposition. In more open construction, vibrated concrete against an exposed clay face would be more effective than grouting, though in practice the extra cost of formwork would have to be taken into account. Did the Authors find, in that particular job, that jacking the concrete was as effective as the grouting ; and were they able to achieve any measurable movement of the concrete ?

With regard to the raft foundation to stanchions Nos. 26, 27 and 28, the Authors had very cunningly overcome the drawback of the raft having no lateral connection, but, in his view, at the sacrifice of some simplicity. The problem of lapping bars where the excavation had to be carried out in short widths was a difficult and constantly recurring one. It might be overcome, in certain instances, by concreting a width narrower than the excavated width, leaving room within the excavation for a lap length of transverse bars and avoiding the necessity of bending them up. Such a method was more applicable to a length of wall or foundation longer than 18 ft., but careful planning was required to ensure that too great a length of earth face was not exposed at any one time. It was quite feasible where, as in the case in point, the main steel ran at right angles to the excavated face, and he felt that it could have been used with advantage in the *Daily News* bui ding, obviating the very tricky operations which led up to the positioning of the grillage.

Anticipating a question which he was sure would be asked—what about prestressed concrete ?—Mr. Severn said he had nothing very constructive to offer that might justify his asking such a question, nor was he thinking in terms of high quality concrete and slender retaining walls. He had in mind the difficulty of carrying (frequently man-handled on restricted sites) 1¼ in. bars bent at right angles with 6 ft. to 8 ft. arms from the nearby thoroughfare to the excavation, negotiating the numerous obstructions on the site, lowering down the hole between many cross-struts, and finally manoeuvring into place between walings and poling boards. There would seem to be a possibility there for the post-tensioning of fairly thick sections, working to stresses of the order used in normal reinforced concrete.

Finally, he applauded the mention in the paper of that part of the construction that was done in brickwork. Although the site was a confined one, he expected it was vast compared with some that the Authors had encountered. Where there was little space for working, there was a real advantage in the use of good quality brickwork for underpinning and retaining walls in preference to concrete.

Mr. BROWN first thanked Mr. Severn for his kind remarks, which were very encouraging.

It was rather difficult, he continued, at a meeting to deal in detail with the lengthy questions that had been raised. But with regard to the methods chosen for the permanent shuttering against earth faces—the in situ concrete skin in the case of the underpinning, and the precast concrete poling boards at the back of the retaining walls—he said that one of the chief difficulties in regard to the approach to the underpinning was the very confined space in which one had to work. The

* *Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on the 28th October, 1954. Dr. S. B. Hamilton, M.Sc., B.Sc.(Eng.), M.I.C.E. (President) in the Chair. Published in "The Structural Engineer", Vol. XXXII, No. 10, pp. 264-273, Oct. 1954.*

Authors had had experience of the jacking method combined with an in situ skin wall before, and they had used it again. That was one reason. The provision for a partially indeterminate load at the back of the underpinning was another factor which had to be considered, and it was necessary to ensure immediate and permanent contact with the earth face to cater for this condition. In using in situ concrete it was considered that this was achieved; this would have been difficult had precast concrete poling boards been used in these confined spaces.

On the question of designing underpinning and, at the same time, obtaining lateral connections, he said that was a perpetual problem, a horrible snag; and when one was deciding what to do there was generally somebody urging one to make up one's mind quickly. The Authors considered that their method was practicable, at any rate, the contractors thought it was reasonable and there were no complaints. The work had to be done in narrow widths and they did achieve a certain amount of connection between the units of underpinning by casting in the temporary horizontal struts, subsequently taken out, thus forming lateral keys. Lateral connections in underpinning are always desirable and alternative methods are always worthy of consideration.

With regard to the manoeuvring of the reinforcement bars Mr. Severn was right in saying that it could be very difficult to handle them in very restricted spaces. Probably one of the best assets that one could have in work of that sort was a good contractor and a man who was accustomed to handling bars; and he was frankly surprised by the ease with which the bars went into their places. Some of them weighed between 3 or 4 cwt. each, and even with the timbering and all the other hazards it did not prove such a terrible job to fix them.

About the suggestion to use post-tensioned prestressed concrete for the retaining walls, Mr. Brown said that for the moment he did not know quite how he would use this. It should be remembered that the contract was under consideration in 1946 and it would be fair to say that prestressed concrete at that time was in its infancy as far as this country was concerned. In any case the clients would have been very suspect at introducing anything chancy: they wanted good solid proven methods.

A speaker agreed with the Authors concerning the importance of good and economic foundation design and said it was not always easy to achieve that, especially in very confined conditions; with the very rapid development of the science of soil mechanics it was appreciated that the ground was a structural material and should be treated as such. All that was very welcome and satisfactory. After the foundations were prepared the building had still to be erected, and a lot of engineering skill was devoted to the bridging of the gap between the drawing board and the site. It was therefore of great value to have such a clear description as the Authors had given of all the operations.

Referring to the construction and timbering of the Bouverie Street retaining wall, the speaker said it seemed that thinnish square timbers were used both vertically and horizontally; although too much timber was better than too little, he would have expected to see the struts a little wider apart, even if they had to be made bigger. He wondered how the skip which the Authors had shown in their illustrations, albeit a small one, would have passed down between the timbers. Perhaps they would say why that particular structure was adopted and whether there were any difficulties encountered during the actual excavation of the trench.

Mr. Brown, after thanking the speaker for his kind remarks, said that originally they had drawn up a scheme for strutting the face at Bouverie Street. It would be appreciated that one of the great difficulties was—and they had emphasised it in the course of presenting the paper—that very heavily laden lorries were passing up and down the street, and even more potent was the fact that the wheels of the lorries could come right to the edge of the top of the wall; there was no margin whatsoever. They had drawn up a scheme for timbering; and bearing in mind that timbering as selected for such a job was not quite so scientific as concrete work, they considered their scheme to be reasonable. It was not acknowledged in the paper that Messrs. Goodman Price had carried out the timbering, but they had considerable experience of such work and executed an excellent task. Many of the men who carried it out were about 70 years old, and he did not think he could argue very much with them.

He did not think there had been difficulty with the skip, although one or two struts had had a clout now and again. It was a ¾-yard skip and, at the rate of excavation attained, its capacity was quite sufficient. The horizontal distance between struts was 8 ft. which left sufficient room to operate the skip. They were not spaced so close as they appeared in the illustration.

Mr. R. A. Inglis (Associate-Member) asked for information on the means adopted to define the safe loads on the foundations.

Mr. Brown said he gathered that Mr. Inglis was referring to the safe ground pressure adopted.

Mr. Inglis replied that he was, and also the means adopted to determine what would be a satisfactory loading.

Mr. Brown said that originally borehole tests were carried out across the site, and some of the materials were subjected to analysis to show their safe carrying capacity. He could not remember the particular details, but 90 per cent. of the building was founded on London blue clay, which from experience was known to be one of the best subsoils in London. With the aid of the knowledge obtained from the tests, and the particular knowledge of the District Surveyor, he being not very far away, the figure of 3 tons per square foot at the general level was arrived at and agreed; and, of course, at still lower levels, where there was even greater compaction of the ground, further tests were made and the loading went up to 4 tons per sq. ft. The loadings adopted were based on tests carried out by one of the testing laboratories.

The building was envisaged in 1946, so that he had not the figures in his memory.

Mr. B. E. Hill (Associate-Member) asked for a little more information on the strutting, the order of the loading placed on it and the actual age of the green concrete.

Mr. Brown said that presumably Mr. Hill was referring to the strutting in the underpinning chiefly.

Mr. Hill agreed.

Mr. Brown said they had, of course, calculated to the best of their ability the pressure to be anticipated from the excavated face under the existing foundation on the *News of the World* side of the site. They could not

get an accurate figure—it would be ridiculous to say that they could—and naturally the jack loads were based on the load which it was anticipated would come from the face. The struts were of steel, 6 in. × 5 in. R.S.J.'s and they could easily take the load expected. He could not quote the figures from memory, but they were not very large, because the underpinning holes were quite small.

The green concrete was jacked back approximately 24 hours after it was cast and due to the conditions obtaining at particular instances further experience was gained as work proceeded. In some cases—mainly due to weather conditions—there were signs of cracking after removal of the forms.

Mr. Frederick S. Snow, O.B.E. (Past President) paid tribute to the Authors on the very good description of a very fine job of work capably handled.

Mentioning one or two features which he considered would amply repay study, he referred particularly to the underpinning of the *News of the World* building. In this connection it was worth while studying the sections reproduced in the paper, noting carefully the underpinning line and comparing the result with what has happened in somewhat similar cases.

Another point he stressed was—and as the Authors had stated—works of this intricate and exacting nature had to be carried out by a skilled contractor. There were large developments proceeding in London to replace the buildings which Hitler had destroyed, but these did not present the same problems in respect of underpinning, as did the building described in the paper, inasmuch as most of these structures were being erected on open sites.

He was very pleased to note that there were present at the meeting the General Foreman, Mr. Aldred, who had run the job and who was throughout the guide, mentor and friend of all concerned with the work; also Mr. Norman Hume of the *News Chronicle* and *Star*, who had helped to see that everything was safe. Undoubtedly the work was executed by a contractor who had a thorough knowledge of the work involved.

On the question of pressures on the timber struts raised by Mr. Hill, he said they were calculated some eighteen years ago and were given in a paper published in the Institution's proceedings, namely " Foundations of London Structures," THE STRUCTURAL ENGINEER, January, 1936, and no doubt the Institution's Librarian could produce the paper which contained the information.

Finally, Mr. Snow emphasized that undertakings of the kind described by Mr. Slatter and Mr. Brown were team jobs; they could not be designed like a reinforced concrete or steel framed structure. The work was carried out by a wonderful team and he congratulated everybody concerned.

Mr. W. Heigh (Member) (Chairman, Scottish Branch) complimented those who had done this work. Although they were fortunate in having subsoil that was not merely the mud and flowing sand with which construction often had to contend, the clean-cut erection procedure looked attractive. But why had they not used a welded girder? In these times, the particular example shown of a large riveted girder seemed extraordinary; never had he seen so many ugly disfiguring knobs; were they really necessary?

Mr. Brown replied that that was an excellent question, coming from a Scotsman!

It would be noticed, he continued, that the whole building was a riveted job. Throughout the work they were up against time. A scheme was drawn up in 1940 and once it had started there was no stopping; there was no time to sit down and choose how to proceed and, indeed, it being a riveted and not a welded job, it followed a sequence. He was sure the client would have been rather perturbed if welding had been used. The newspaper people were particularly fussy about their buildings and would not stand for anything they did not know much about.

Of course, it could have been a welded job.

Mr. S. M. Reisser (Member) asked whether the girder shown on one of the slides and having large openings in the web was designed as a Vierendeel or plate girder.

Mr. Brown replied that it was tried as both.

Mr. F. M. Bowen (Associate-Member of Council) referring to the underpinning as shown in Fig. 4, said it seemed to him from that detail that it must have been extremely difficult to get into place and consolidate the top few inches of concrete, and he asked whether in fact any great difficulty was experienced in that connection.

In the illustrations and in the paper generally he could not find anything about waterproofing the building, and he invited the Authors to comment on their approach to that matter.

Mr. Brown replied that there was no particular difficulty about placing the last 3 ft. of concrete. The audience might have visualised that the concrete went in with rather a large slump but such was not the case. In carrying out the work they were, of course, very rigid about that and there was continual supervision as there would be always in a job of that type. The outside shuttering went up a board at a time and the last few inches prior to pinning was virtually spooned in. The actual underpinning operation was carried out by ramming a mixture of half dry 1 : 1 sand and cement mix, which filled in any voids existing on the top of the concrete that had been put in.

With regard to waterproofing, he said they did use an additive in the concrete mix below ground level. There was not a very serious water problem; the ground tests did not show standing water there which for example would fill the basement, and they considered that good quality concrete would give a watertight condition. So long as there were clean and properly made joints there would not be much moisture through the concrete. That was the answer in that particular job; but, of course, he would not say that it was the answer to every job.

Mr. F. N. Chapman (Associate-Member) felt that it should be placed on record that the brickwork was laid dry. It was the first time he had seen that done, and he asked what procedure was followed to ensure that the bricks were in proper contact with each other—any unevenness of the surfaces would produce a tendency for one brick to ride on another and for certain pressures to arise.

Speaking of the jacking of the green concrete skin wall, he said presumably there was not sufficient time for the concrete to harden before the earth tended to move into perfect contact with the wall; otherwise if the wall were allowed to harden there might have been less likelihood of it coming into action before it was matured.

Thirdly, referring to the bars in the retaining wall, he said the wall was 6 ft. thick, and he wondered whether it would have been possible to have arranged 1 in. dia. bars in two or three rows instead of one row of 2 in. dia. bars.

Mr. Brown replying to the question concerning the dry brickwork, said that ostensibly that was used as temporary part underpinning, because they had wanted to take it out at a later date, to enable the stanchion grillage to be inserted before the final pinning up could be done. The bricks were, of course, engineering bricks. They were very regular and there was a sprinkling of sand to ensure that they sat down well.

Mr. Chapman said that presumably the bricks had no frogs and that the work was not done at the normal pressure.

Mr. Brown replied that the bricks were not completely frogless, but had a very shallow rectangular frog.

The President asked if the frog were filled.

Mr. Brown said that sometimes, when they were not getting a very regular pattern, sand was put down to give a reasonable bed.

As to the jacking back of the green concrete skin wall before it was mature, that was an instruction to ensure that the concrete skin wall forming the earth support was put in close contact with the earth face. The fact that the concrete was green and therefore flexible enabled this to be done with greater assurance. The arrangement had the additional advantage that underpinning could proceed at maximum speed.

The bars used were of a size which was thought to be the optimum. There were 2 in. diameter bars in the main foundation. These had been arranged with a complete lap at the stem of the wall and if they had used a greater number of smaller bars, they would have encountered difficulties with the bar arrangements and the whole of the wall would have been rather congested by a large number of slenderish bars. The 2 in. diameter bars and spacing of same were arranged to meet the timbering detail for the vertical face.

Mr. S. Vaughan (Vice-President), referring to Fig. 4, said it showed the job in the condition that the upper strut had been jacked and the lower strut was in course of being jacked. He asked if the Authors could describe the sequence of the operations which followed, namely the building up of the concrete underpinning, with special reference to the time when each successive frame of timbering was removed ; and also whether any special precautions were taken to maintain the pressure which had been exerted at an earlier stage against the earth face by jacking, during the removal of the struts and the casting of the concrete.

Mr. Brown said it was difficult to show on the detail the sequence of concreting operations that followed the jacking and excavation operations. The holes dug were timbered in the normal way for excavations generally, on three sides. Inside the main excavations there were horizontal timber struts, to the side of which were attached the steel struts with the jack in between.

The jacks were then operated to a predetermined load and following this, the timber struts were wedged until the loading on the gauges had reduced to zero. The steel struts and jacks were then removed to be used on subsequent frames which were carried out similarly to the foregoing until the full depth was reached.

When the jack came out, the waling shown at the back also came out, and the main timber struts were taking their reaction from vertical timber posts at each corner of the hole which held back the concrete skin wall. As the underpinning concrete was brought up, the vertical and horizontal timber struts that were left in at that juncture were wrapped with felt, and concreting took place all round them as shown in the drawing. This procedure was adopted until each hole was complete ; the timber struts left in during the early stage were withdrawn as the adjacent holes were dug, thus leaving a key in the side for the next section of concrete.

Mr. Slatter, who was asked if he wished to add any further remarks, said that if, on reviewing the record of the discussion, the Authors found that the questions had not been dealt with satisfactorily they would endeavour to enlarge on them in writing.

Mr. N. Hume (*Daily News*) said on behalf of his organisation that they considered Messrs. Snow & Partners had done an excellent job of work. They were very satisfied with all that was done. (Applause.)

The President formally expressed thanks to the Authors for the very able manner in which the questions had been answered. They had stood up to the bowling very successfully and the Institution was grateful to them for having answered so informatively.

Book Reviews

(*continued from page 95*)

Constructional Steelwork Shop Practice : A Text Book for Apprentices and Students, by John Farrell. (London : Iliffe, 1953.) 214 pp., $8\frac{1}{2}$ in. × $5\frac{1}{2}$ in. Price 15s.

The British Constructional Steelwork Association are to be congratulated in being instrumental in the production of so comprehensive and practical a text-book. Although a book designed for the use of apprentices and students its value and interest is by no means confined to them, and it is well worth a place in the reference library of the practising engineer.

The well written text is supplemented by an unusually large number of working drawings and diagrams which allows the various processes to be followed with complete understanding by the apprentice or student and, at the same time, the designer is given an insight into the processing of his material which cannot fail to reflect in his design. He is, in fact, shown clearly the problems and methods of the man who actually handles the material.

R. J. V.

The Mechanics of Engineering Soils, 2nd Edition, by P. L. Capper and W. Fisher Cassie. (London : Spon, 1953.) 315 + xvi pp., $8\frac{3}{4}$ in. × $5\frac{3}{4}$ in. Price 25s.

Owing to the rapid development of soil mechanics it has been found necessary to issue a new completely revised edition of this useful book. Several sections have been rewritten, including those on shearing resistance, bearing capacity, pavement design and flow nets, and a new section has been added on soil suction. In addition there are some new examples and illustrations and the bibliography has been extended.

The book is well produced and can be recommended as giving a clear and concise account of the theory of soil mechanics and its application to engineering.

Some Economic Aspects of Single Storey Shed Design*

Written Discussion on the Paper by Mr. R. P. Haines, O.B.E., A.M.I.C.E., A.M.I.Struct.E.

MR. K. C. DAVEY (Associate-Member) : On the basis of the graphs shown in Fig. 10, I have found it interesting to extract some figures and consider them further, to arrive at a relative cost relationship between them as opposed to the purely weight relationship. I append below in tabulated form comparative cost and weight figures for trusses of spans considered by the author at spacings varying between 12 ft. 6 in. and 20 ft. in increments of 2 ft. 6 in. The cost is expressed in the form of equivalent units and for this purpose I have assumed that the relationship between the prices of individual components of the building is as shown below.

Item	*Comparative Unit of cost per ton*
Purlins	1.00
Stanchion & Valley Tie ..	1.18
Roof Truss & ½ Point Tie	1.45

While the above relationship may not be strictly accurate, since I have not any recent information regarding ruling prices in the United Kingdom, I consider that they are sufficiently so to illustrate my point.

In cases where a step occurs in the graphs of Fig. 10 I have taken the lower figure for the weights shown in the table.

38 ft. SPAN

Item	12′ 6″		15′		17′ 6″		20′	
	Wt.	Cost	Wt.	Cost	Wt.	Cost	Wt.	Cost
Purlins - - - -	1.29	1.29	1.50	1.50	1.73	1.73	1.73	1.73
Stanchion & Valley Tie -	0.84	0.99	0.81	0.96	0.82	0.97	0.76	0.90
Roof Truss & ½ Pt. Tie -	1.68	2.42	1.54	2.23	1.40	2.03	1.35	1.96
Total	3.81	4.70	3.85	4.69	3.95	4.73	3.84	4.59

48 ft. SPAN

Item	12′ 6″		15′		17′ 6″		20′	
	Wt.	Cost	Wt.	Cost	Wt.	Cost	Wt.	Cost
Purlins - - - -	1.18	1.18	1.39	1.39	1.60	1.60	1.60	1.60
Stanchion & Valley Tie -	0.67	0.79	0.65	0.77	0.67	0.79	0.60	0.71
Roof Truss & ½ Pt. Tie -	1.70	2.47	1.71	2.48	1.62	2.35	1.68	2.44
Total -	3.55	4.44	3.75	4.64	3.89	4.74	3.88	4.75

58 ft. SPAN

Item	12′ 6″		15′		17′ 6″		20′	
	Wt.	Cost	Wt.	Cost	Wt.	Cost	Wt.	Cos
Purlins - - - -	1.10	1.10	1.30	1.30	1.50	1.50	1.50	1.5
Stanchion & Valley Tie -	0.56	0.66	0.50	0.59	0.53	0.63	0.56	0.6
Roof Truss & ½ Pt. Tie -	1.96	2.84	1.85	2.68	1.80	2.61	1.77	2.5
Total -	3.62	4.60	3.65	4.57	3.83	4.74	3.83	4.7

68 ft. SPAN

Item	12′ 6″		15′		17′ 6″		20′	
	Wt.	Cost	Wt.	Cost	Wt.	Cost	Wt.	Cos
Purlins - - - -	1.07	1.07	1.28	1.28	1.48	1.48	1.48	1.4
Stanchion & Valley Tie -	0.47	0.55	0.52	0.61	0.52	0.61	0.48	0.5
Roof Truss & ½ Pt. Tie -	2.08	3.02	1.96	2.84	1.93	2.80	1.86	2.7
Total -	3.62	4.64	3.76	4.73	3.93	4.89	3.82	4.7

Considering the four sets of figures for each span o truss the percentage increase of the highest over th lowest figures for weight and cost are as follows :

Span	*Optimum Spacing*	*Percentage Increase in Weight*	*Percentage Increase in Cost*
38 ft.	20 ft.	3.67	3.05
48 ft.	12 ft. 6 in.	9.58	6.98
58 ft.	15 ft.	5.80	3.04
68 ft.	12 ft. 6 in.	8.56	5.38

It can be seen that in each case the apparent increas in cost from varying the truss spacing from th optimum value is less than the similar value for weight Except in the case of the 38 ft. span where there is a apparent economy in increasing the truss spacing, th cost increases with the increase of spacing. However if the economy of the building is considered as a whol there is a saving to be achieved in increased spacing by (*a*) there being a reduced number of foundation block and column casings and (*b*) a possible economy in th price of erection. Further, since the cost of the super structure represents only 31 per cent. of the total cos the variation in cost of steelwork is of greatly reduce significance.

If economy in weight is the main consideration a spacing of 12 ft. 6 in. appears to be best, but if a sacrifice in weight can be made then I submit that trus spacing has a negligible effect on the overall economy of the building.

* Published in *The Structural Engineer*, Vol. XXXII, No. 5, pp. 135-152 (May 1954).

While not suggesting that an increased spacing is more economical I do feel that, all things being considered, the slight (if any) additional cost can be outweighed by the advantages gained, particularly in the case of a multi bay storage or factory building.

In considering portal frames it occurs to me that it might be worth considering the adoption in Case 4 of Fig. 13 of a similar device to that described for Case 1 where a deliberately induced eccentricity of 6 in. in the vertical reaction could give a modified B.M. at a point 1 ft. 6 in. from the haunch of 46,750 lbs./ft. This revised figure is then closer to that of 42,000 lbs./ft. quoted for Case 3, and although it is still considerably in excess of the maximum moment in Case 1 it does render the two pinned frame a more attractive proposition where the space under the roof is required for either storage space or accommodation of runway beams.

Mr. E. J. Buckley (Member) : Mr. Haines is to be congratulated on the unusually informative paper. Of particular interest is the very detailed treatment of purlin design. The relative importance of the weight and cost of purlins is frequently overlooked and the suggested modes of design are rational methods of effecting considerable savings.

If some criticism might, however, be permitted on the broader aspects, the following comments could be made :

(*a*) The study of the economies of this type of building is very complex and cannot, rationally, ignore the functional requirements of the finished structure. Shortages, artificial or otherwise, of strategically important basic materials such as steel, have led to forms of construction and structural concepts which, whilst effectively giving weatherproof coverage to a given floor area, are certainly not structurally efficient.

The analysis of the efficiency and, thus, the true economy of any scheme must involve the careful consideration and balance of at least the following items :

(i) Capital cost ;
(ii) Maintenance charges ; and
(iii) Functional efficiency.

In very many instances the structure for which the lowest capital expenditure is required gives the lowest efficiency under heading (iii) and represents a very short-termed economy.

(*b*) In this country, and even in relatively " cheap labour " areas overseas, mechanical handling of merchandise or other goods is becoming increasingly common and with it has come a demand for far greater unrestricted floor areas. Structures fulfilling these conditions can seldom be the cheapest method of giving coverage but, nevertheless, do represent the most efficient and therefore, ultimately, the cheapest form of construction when considered on the basis of (*a*) above.

A glance at Fig. 11 of the paper will show how very unsuited is such a building, with its closely spaced internal stanchions, should it be proposed to employ, say, fork lift trucks for stacking goods. Further, it must be remembered that each internal stanchion and the floor in the immediate vicinity represents a potential " dead " area for storage purposes.

It is mentioned in the paper that intermediate internal stanchions could be omitted if suitable valley girders were provided. An alternative, however, that should not be overlooked is greatly to increase the spacing of the trusses and to employ some form of light welded tubular lattice purlins or, possibly, lattice purlins built-up from one of the many proprietary pressed steel sections. This method of reducing the otherwise large number of roof trusses can be, structurally, very satisfactory in very large multi-space storage buildings.

(*c*) The conventional method of roof valley drainage proposed in the paper can be greatly improved under certain circumstances. Very large sheds have been successfully drained by the provision of pressed steel valley gutters of non-uniform cross-section, conducting the rainfall to gable rainwater pipes at each end of the building, thus permitting the omission of the internal downpipes and the underfloor drains both of which, at least in some cases, have considerable disadvantages and, in any case, are very expensive.

Mr. R. A. Sefton Jenkins (Associate-Member) : The Author states that aluminium has not been considered for short spans owing to its high cost. This statement can be very misleading without any qualification.

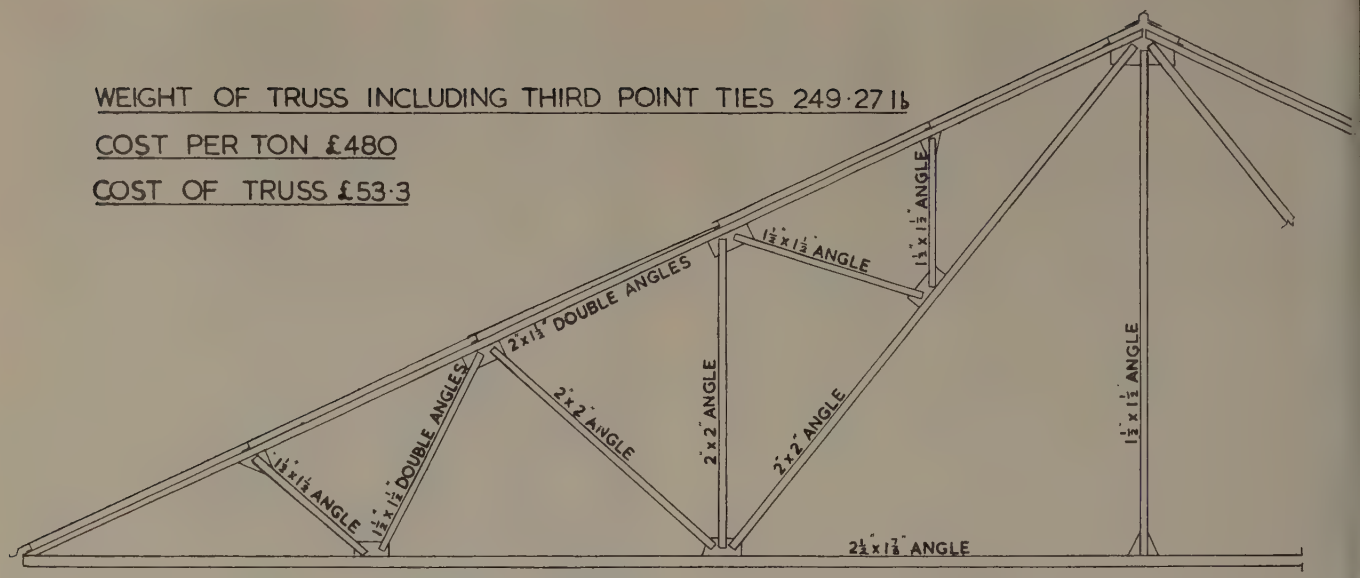
I suspect, from the way in which he has used standard angle sections for the examples he gives of purlins, that in common with a great number of engineers he believes that aluminium is merely a light-weight steel with slightly differing properties. This philosophy has not been helped in the past by the publication of B.S.1161 which gives standard sections very similar to steel sections. A designer must appreciate fully the potentialities and restrictions of the material he is using. No one would consider making a beam of steel and a beam of timber of the same shape, and yet the physical characteristics, the methods of manufacture, and methods of fabrication of steel and aluminium are as widely different as those of steel and timber.

In his example of aluminium purlins (Fig. 6) he shows a B.S.1161 4 in. × 3 in. × $\frac{1}{4}$ in. angle. Recently the Aluminium Development Association have published a series of lipped angle and channel sections which are a great improvement on B.S.1161 sections. A standard lipped channel of the same section modulus would be $3\frac{1}{2}$ in. × $1\frac{3}{4}$ in. × 0.109 in. thick weighing 1.07 lbs./ft. or 3/6d. per ft., taking a slightly higher cost per lb. than that used by Mr. Haines. This is only slightly more than the cost of steel purlins and cheaper than tubular purlins given in Fig. 6.

Recently I designed some aluminium buildings of 33 ft. span ; for various reasons it was more economical to place the trusses at 15 ft. centres, and the purlins at 10 ft. centres carrying aluminium sheet. The purlins weighed 1.74 lbs./ft. or at 3/6d. per lb. just over 6d. per sq. ft. The cost of steel purlins as given by Mr. Haines in Fig. 6 is $8\frac{1}{4}$d. per sq. ft. The trusses (33 ft. span, 15 ft. centres) weighed 168 lbs. or at 4/5d. per lb. 1/6$\frac{1}{4}$d. per sq. ft. The cheapest steel truss (48 ft., 12 ft. 6 in. crs.) that Mr. Haines considers is £30.4 or 1/0d. per sq. ft. the most expensive is £39.6 or 1/4d. per sq. ft. The reinforced concrete truss is £49 or 1/7$\frac{1}{2}$d. per sq. ft.

	Truss Cost/sq. ft.	*Purlin Cost/sq. ft. (based on Fig. 6)*	*Total*
Steel, cheapest 48 ft. × 12 ft. 6 in.	1/0d.	$8\frac{1}{4}$d.	1/8$\frac{1}{4}$d.
Steel, most expensive 48 ft. × 12 ft. 6 in.	1/4d.	$8\frac{1}{4}$d.	2/0$\frac{1}{4}$d.
Concrete 48 ft. × 12 ft. 6 in.	1/7$\frac{1}{2}$d.	$8\frac{1}{4}$d.	2/3$\frac{3}{4}$d.
Aluminium alloy 33 ft. × 15 ft.	1/6$\frac{1}{4}$d.	6d.	2/0$\frac{1}{4}$d.

Whilst it is probable that a 48 ft. aluminium truss would cost slightly more per sq. ft., it can be seen from the above comparisons that an aluminium structure can be built for approximately the same or slightly more than a steel structure, and usually cheaper than concrete.



Conventional HE IOWP Aluminium Riveted Roof Truss
48′-0″ Span

Fig. 1

When this is coupled with the great advantage that aluminium requires little or no maintenance, it is surprising that more aluminium structures have not been built. The reason is, I feel, mainly the fault of engineers who have not gone deeply enough into the characteristics of aluminium but have tried to design aluminium as though it were steel. They have then been surprised that it works out expensive.

Whilst there is still a great deal of practical research to be carried out, not only on the design side of aluminium structures, but also into the production and fabrication techniques, aluminium will, I am sure, take its place as a structural material in the same way as steel, prestressed steel, reinforced concrete and prestressed concrete have done.

Mr. M. Bridgewater : Economic comparisons of different structural designs are always difficult to make, and Mr. R. P. Haines' paper will prove valuable when single-storey sheds are under consideration. In examining various types of 48 ft. span trusses, the Author omits to consider aluminium on the grounds that it is too expensive for spans under consideration. Whilst it is true that, due to the high ingot price, aluminium structures only become directly competitive for long spans, the price for a 48 ft. span truss is not large and would, in many cases, be offset by savings in maintenance costs, which could be realised by the durability of the aluminium.

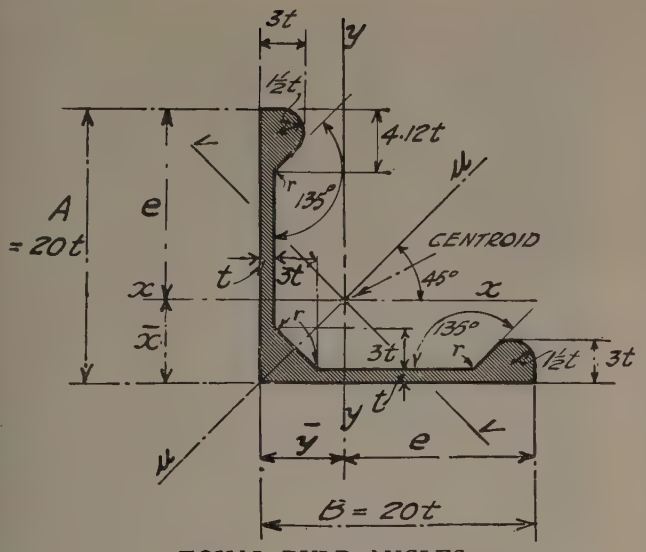
Fig. 1 shows a 48 ft. span HE10WP aluminium truss design for 11 ft. 6 in. centres and for superimposed snow and suction loads of 10 lb./sq. ft. The sections used throughout this design are a range of bulbed angles (Fig. 2) that has been specially developed by the Aluminium Development Association for efficient structural design. By the utilisation of thin extruded sections with bulbs and a fillet to prevent the onset of torsional instability, a great saving in structural weight can be obtained over the more conventional plain sections.

The cost of the aluminium truss would be approximately £54, as compared to £40 for the conventional steel truss, whilst its weight is only a quarter that of the latter. A conservative estimate for repainting 48 ft. steel truss would be about £7, so that, in an atmosphere which necessitates regular painting, the aluminium truss would appear to be an economical proposition, particularly if such maintenance resulted in interruption to production.

If the lipped channel section shown on Fig. 2 is used for purlins instead of the more conventional plain angles, the prices of aluminium purlins would be considerably reduced. A $3\frac{1}{2}$ in. × $1\frac{3}{4}$ in. lipped channel weighing 1.07 lb./ft. run would have the same section modulus as the 4 in. × 3 in. aluminium angle shown in Table 6 of the paper and would reduce the cost of the purlin from 6/6d. to 3/4d./ft. At this price, aluminium becomes nearly competitive with the other types of purlin considered and, again, any necessity for maintenance would make it a definite economic proposition.

Due to its low modulus, aluminium purlins have high deflections and experience has shown that it is this factor, rather than bending stresses, which governs the design. Advantage cannot therefore be taken of the plastic method advocated in the paper, as this would result in excessive deflections. The $3\frac{1}{2}$ in. × $1\frac{3}{4}$ in. lipped channel, continuous over two spans, would limit the deflection to $\frac{\text{Span}}{180}$, a figure which has been found acceptable in practice and the price/ft. for an aluminium purlin would remain at 3/4d. instead of the 5/8½d. given in Fig. 7 for the $3\frac{1}{2}$ in. × $2\frac{1}{2}$ in. × $\frac{1}{4}$ in. angle.

Captain J. H. Foster : In expressing my views on Mr. Haines' paper I should mention our long and most friendly association during my past service, and acknowledge the help he gave me in my efforts to prolong the life of the War Departments steel structures. I hope it is possible for me to suggest that his views and my own were at all times in complete accord. It is clear from the paper that his Department has given all types of single storey shed construction very full consideration and the facts stated will certainly convince any reasonable man that steel construction is the economic answer for W.D. single storey shed construction.

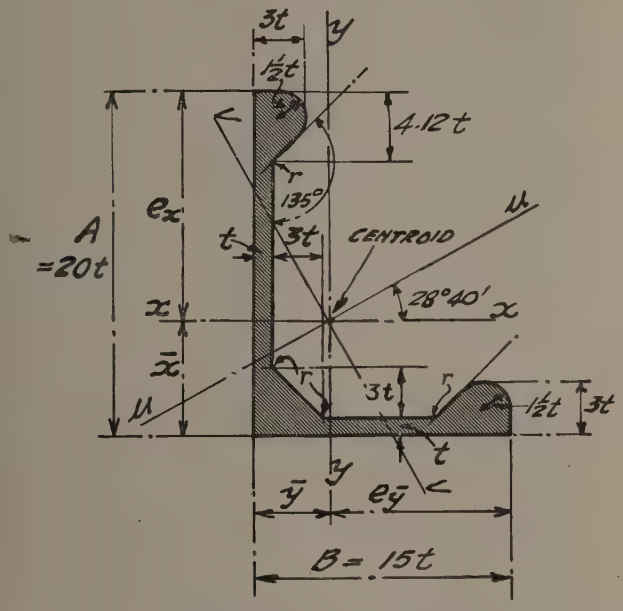


EQUAL BULB ANGLES

$r = 2t$

t	= thickness
a	= sectional area = $54.9t^2$
w	= weight in lb. per ft. = $1.215a$ (a in in.^{2})
$\bar{x}$ & $\bar{y}$	= $6.07t$
I_x & I_y	= $2606t^4$
I_u	= $4011t^4$
I_v	= $1201t^4$
k_x & k_y	= $6.9t$
k_u	= $8.55t$
k_v	= $4.68t$
Z_x & Z_y	= $I/e = 186t^3$
J	= torsion constant = $51.9t^4$

PROPERTIES OF SECTIONS

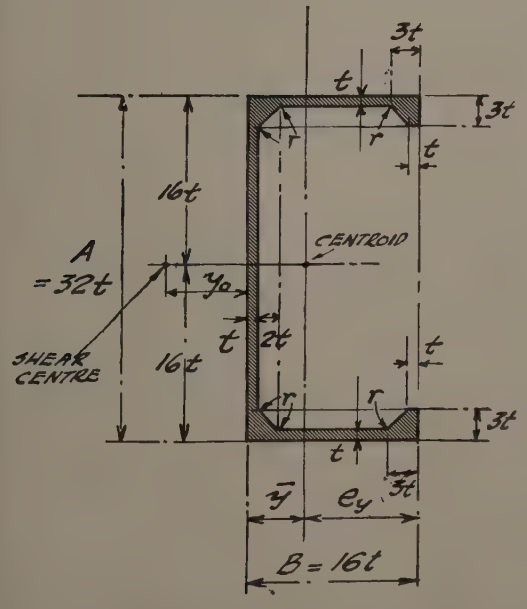


UNEQUAL BULB ANGLES

$r = 2t$

t	= thickness
a	= sectional area = $49.9t^2$
w	= weight in lb. per ft. = $1.215a$ (a in in.^{2})
$\bar{x}$	= $6.62t$
$\bar{y}$	= $4.35t$
I_x	= $2430t^4$
I_y	= $1234t^4$
I_u	= $2940t^4$
I_v	= $723t^4$
k_x	= $6.98t$
k_y	= $4.97t$
k_u	= $7.67t$
k_v	= $3.81t$
Z_x	= $I_x/e_x = 182t^3$
Z_y	= $I_y/e_y = 116t^3$
J	= torsion constant = $50.2t^4$

PROPERTIES OF SECTIONS



LIPPED CHANNEL SECTIONS

$r = 2t$

t	= thickness
a	= sectional area = $74t^2$
w	= weight per ft. = $1.215a$ (a in in.^{2})
$\bar{y}$	= $5.37t$
I_x	= $12{,}360t^4$
I_y	= $2401t^4$
k_x	= $12.92t$
k_y	= $5.70t$
Z_x	= $I_x/e_x = 772.5t^3$
Z_y	$I_y/e_y = 22.59t^3$
J	= torsion constant = $41.6t^4$
y_o	= distance to shear centre = $6.64t$

PROPERTIES OF SECTIONS

Fig. 2

It is perhaps fortunate that Mr. Haines concludes his paper by a reference to the human factor, and his last word is " hope." Singularly enough I was also able to conclude my previous work on the same note via a different route—Shell concrete. It may be of some interest to mention that I was once reminded that the moral was to the material as three to one, and in the circumstances I felt it necessary to prove the contrary could also be true ; relating the ' proof ' of this latter proposition using shell concrete in the abstract is not a course open to me.

In considering what might yet be done to enable concrete to compete more effectively with steel for Service requirements of single storey shed construction, I am glad to note that one of my subsidiary schemes, corrugated concrete walls, will be tried out on an economic scale. Perhaps I can suggest that this is putting barrel vaults on their beam ends ? When I think that in my first relationship with Mr. Haines we were in accord on ' shell ' afloat, and that he will probably agree that we are in accord on ' shell ' submerged, is it reasonable to forecast that he might eventually also prove that shell concrete construction is the economic answer for W.D. single storey shed construction ?

For Mr. Haines even to consider shell concrete again, the shell concrete field will have to take fresh thought, and consider how the cost of shell concrete roof construction can be reduced. If the W.D. find shell concrete walls economic, I suggest to the shell field that in extending the principle to roof construction there are certainly grounds for hope.

Some mention has been made elsewhere of my attempts to use gun placement of concrete on permanent formwork to achieve greater economy in shell construction. My view has always been that with well organised site operations, gun placement can be much less costly. Possibly the companies in the gun placement field might say that they have never had an opportunity to show what they can do ; now Mr. Haines very kindly provides ample scope for them.

The use of gun placement in Ctesiphon construction is the way parity in cost with steel construction can be obtained in single storey shed construction. I have always said that the exponents of this type of construction, Major J. H. de W. Waller, D.S.O., O.B.E., and Mr. A. C. Aston, have not done justice to themselves in stating their case. Instead of determining the longest feasible span, which I understand is several hundred feet, I would ask them to consider what is the lowest cost for covered area in Ctesiphon. Even with an arbitrary layout of 50 ft. × 25 ft., with the Ctesiphon Shells arching between the 50 ft. valley beams, a strong case was made out by me, towards achieving parity with steel construction. There probably is an optimum layout for multispan Ctesiphon construction which can be derived from the best combination of scaffolding and placement operations. I suggest that when this question is clarified, and the Companies concerned with gun placement are given further training by the W.D., in wall construction, an economic case for single storey shed construction in shell concrete will be established.

The next step would be to take some action ; this should be possible, if further expanses of covered accommodation is required by the W.D., in competition with steel construction.

I have made and am still making attempts to compete in cost with Major Waller's Ctesiphon construction ; and it is clear that as far as my efforts are concerned with gun placement they are derived from his original concept, and M. Monier's. Major Waller and I seem also to be on common ground with mud bricks.

If I have left the shell concrete field some hope for their future progress, (I assume Ctesiphon construction is their main competitor), I can ask them to thank Major Waller, and Mr. Haines.

In conclusion my thesis is that if the shell field really got down to earth, they could compete with lined single storey shed construction. However, for unlined sheds, I see no immediate answer except Major Waller's.

My hope is that the shell field will now stand up and compete more effectively against steel construction, and not shoot at their friends ; if guns are to be used let them be directed at Mr. Haines' walls. I should ask them not to use rocket projectors which blow backwards as well as forwards.

It may well be that my old friend Mr. Haines will so revise his views that he will one day insist on square base domes being used for his sheds ; when he is ready I hope the shell field will also be ready for action, and that he can be sure of shell.

Professor R. G. Robertson (Member) : The writer thought that the use of curved rafters, as in the Dutch Barn, would lead to economy, especially if prestressed tie rods were used, thus eliminating all dead load moment in posts and rafters : had this been considered ?

This type of roof had been mentioned already by the Author in regard to the economy in purlins but not in regard to the economy in rafters.

Reply to written correspondence

Mr. Davey's criticism of the comparison of weights, on the grounds that the related costs do not necessarily follow the same trend, is to a small extent justified ; and he is to be complimented on the very detailed tabulation he has prepared. I must point out, however, that the weights in Fig. 10 are based on the use of purlins much lighter than would normally be employed if B.S.449 were used as a basis for design, and that had these B.S. purlins been used a far greater weight disadvantage would have resulted at the larger truss spacings. As it is, the labour cost for fabricating the light section purlins used is very little different to that for heavier conventional size purlins, so that it would be reasonable to expect the rate per ton to be rather higher than normal and a relation of 1 to 1.2 might be nearer the truth in this case for the purlin to truss rates than the 1 to 1.45 assumed by Mr. Davey. In addition for truss spacings of over 17 ft., the purlin weight used in Fig. 10 allows for the use of 4 in. × $1\frac{3}{4}$ in. joists with sag rods so that either the extra fabricating cost of such a construction must be taken into account or else a weight more consistent with laterally unsupported purlins must be used.

The cost of foundations is so small in comparison to the cost of the frame that the greater incidence of foundations would have a negligible effect on the economics of the spacing.

With Mr. Davey's last point I am inclined to agree—eccentric pins could easily be introduced at the feet of the two pinned frame to make the moments nearer to those in the fixed version.

Mr. Buckley very rightly considers maintenance charges and functional efficiency as of equal importance to capital cost, and it is freely admitted that, in particular, functional efficiency must take priority over

structural economy in dictating optimum spans and stanchion spacings. The Author has, however, known cases where the use of expensive construction has been justified by the large spans they achieved when the function of the building really required no such provision. It is of course more than possible that the spacious effect achieved might have a beneficial effect on production within such a shop and also that, in years to come, the relatively unobstructed floor space would facilitate any change of function that might occur during the life of the building.

In the case of the buildings mentioned in the paper —48 ft. and 60 ft. spans—the ' Users ' were of course consulted and their requirements were fully met by the spans and spacing adopted. The buildings were in fact planned round a precise stores stacking layout and a system of fork lift truck access aisles which were found to be required for storage, issue and inspection, after a detailed investigation by the W.D. stores organisation. In the 48 ft. span design mentioned by Mr. Buckley, movement was to be through the building from gable to gable and no traffic between adjacent spans was required. The valley stanchions were therefore placed at 12 ft. 6 in. centres. In the 60 ft. span shed where access between bays was required a 25 ft. spacing of valley stanchions was used.

The use of latticed purlins and triangular profile space frames formed by latticing together adjacent purlins had been given some consideration but appeared to show no advantage, although it is probable that this form of construction will have a great deal to commend it when truss spans are large.

Upon reflection I feel I have perhaps been guilty of being too conventional in the methods used for roof drainage, and from a check recently made, it appears that Mr. Buckley's plea for large girth gutters capable of carrying the roof water from gable to gable without intermediate downpipes has the merit of overall economy whilst incurring far less disturbance if sub soil drains have to be uncovered for any reason. The thought of using a gutter of differing section does, however, disturb me.

In reply to Mr. Sefton Jenkins, I must admit that I have perhaps rather too lightly dismissed aluminium ; but I hasten to assure him that I do not subscribe to the philosophy of the material being a light weight steel. The wide variety of shapes made possible by the extrusion process is as well known as the limitations imposed by the rolling process in steel manufacture. But the engineer might be forgiven for being a little hesitant about adopting purpose made sections despite the knowledge that the dies can be cheaply produced, and there may therefore be a tendency to utilize sections which are known to be both readily and universally available. It is therefore of great interest to learn that the range of standard sections has been increased beyond those listed in the B.S. to include structural extrusions designed to enhance the section properties by a better distribution of the metal.

The weight Mr. Sefton Jenkins has put forward in justification of aluminium for short span roofs shows such a remarkable economy in the use of the material that it would stand reasonable comparison with a steel equivalent. It is therefore to be regretted that the particular example he has chosen has so few points of similarity with the steel construction with which he has compared it. He may perhaps concede that a 33 ft. span truss in aluminium with 3 lines of purlins has little in common with a steel roof of 48ft. span with 10 lines of purlins particularly as the 10ft. centres of purlins in the aluminium design can only be achieved at the expense of the roof cladding.

Mr. Jenkins has given the price of his aluminium construction at 2/0¼d. per sq. ft. The comparable construction in conventional bolted steel works out at 1/6¼d. per sq. ft. made up as follows :

Truss 1.44 lbs. sq. ft. @ -/9.3d. = 1/1½d. and Purlins — 3 lines — 0.5 lbs. sq. ft. @ -/9.5d. = -/4¾d.

I am a little diffident about adopting the suggestion of a 3½ in. × 1¾ in. lipped channel section of 11 gauge metal for a purlin of 12 ft. 6 in. span in view of the deflection characteristics of aluminium (roughly three times that of steel), and feel that some difficulty would be experienced in erecting the roof sheeting unless sag rods were used to restrict the lateral deflection. My fears may be groundless but I remain to be convinced. In an article I wrote some six years ago, I note I came to the conclusion that a beam of aluminium required to be 50 per cent. deeper, and to have flanges 33 per cent greater in area than the equivalent steel section if equal deflection for both was required.

I agree with both Mr. Sefton Jenkins and Mr. Bridgewater in their contention that aluminium does make a saving in maintenance cost, and I am particularly interested in the very slender design that Mr. Bridgewater has produced. My first reaction to the construction is that despite the fact that the War Office design is as light as most contemporary conventional steel designs, there would appear some room for improvement when one considers that in aluminium 1½ in. riveting legs are being adopted and presumably ½ in. diameter rivets are utilized. The cost comparison is very illuminating and shows that by this very keen design and the use of bulb angles the light alloy can get on to terms with other mediums of construction when long term maintenance is considered—even though Mr. Bridgewater's figure of £7 for painting and truss might be disputed.

Capt. Foster has for many years preached the case for shell concrete barrel vaults ; but with the possible exception of Ctesiphon—which uses little formwork in its construction—I have yet to see figures based on reliable calculation from the experience of jobs actually built which show the barrel vault to be less than 50 per cent. greater in cost than steel construction, or much better than parity in total usage of steel.

Although that less conventional gunned shell construction might possibly get a little nearer, such construction is as yet untried and any cost forecasts are likely to be rather in the nature of hopeful optimism. Capt. Foster's latest plea still leaves me unconvinced and unrepentant.

Indeed, when the two constructions are compared it is not surprising that the traditional barrel vault is dearer since it gives a durable low maintenance finish, a heavy impervious roof and a pleasing architectural form, and it is admitted, in every way, gives good value for money for jobs where a little more expenditure is justified. The War Department requirement was, however, for the cheapest form of industrial building.

I do not wish to put forward yet further sets of figures in justification of steel ; but a simple comparison will serve to illustrate the position that steel framed asbestos clad structures stand in relation to concrete vaults. The whole of the steel framing of a shed costs about 3/0d. per sq. ft. on plan, erected and painted. The cost of the formwork and shuttering to a barrel is about 2/6d. per sq. ft. on plan. This therefore leaves the reinforcement concrete and surface weatherproofing

to be provided and placed at nearly the same price as asbestos sheeting—a state of affairs which is unlikely to be realised very easily.

Replying to Professor Robertson's point regarding curved rafters, I have not considered the arch type of roof to any great extent. The ordinary arched latticed truss has some small economy over the pitched truss due to the reduced forces in the rafters and main ties, but it suffers from the disadvantage of extra fabrication in curving the rafter, slightly longer internal members, and more expensive curved types of sheets are required for the roofing, and reduced weather protection is obtained at the horizontal laps near the crown. The pre-war Dutch barn roof was of course in a category of its own, it was extremely light and it was never clear whether the truss and purlins carried the roofing sheeting or whether the reverse happened; whatever the action, both structure and cladding leaned happily on each other and mutually contrived to stay up. In some very early examples of this type of roof the trusses as such were dispensed with altogether, and light radial tie rods were bolted to the cladding to retain the sheeting in the arch profile.

As regards the use of prestressed arches, long span roofs have been constructed in Belgium of concrete arched ribs post tensioned along the rib axis and then further stressed by a horizontal cable joining the shoes to induce a reverse bending against the self weight of the arch and the precast concrete slab roofing. Such structures are of course substantial and expensive, but if prestressing is applied to a steel arch little advantage accrues since the superimposed load is normally the major load to be carried in this case, and in arches the condition of snow and wind loading to one side produces positive and negative moments on either side of the span of roughly equal intensity so that pre-stressing across the shoes would merely serve to worsen the loading condition.

I have roughed out the size of an ordinary arch rib for a 48 ft. span roof of 11 ft. rise, and it appears that a 5 in. $\times$ $2\frac{1}{2}$ in. R.S.J. would suffice for the rib with a $\frac{3}{4}$ in. dia. M.S. tie rod connected across the shoes. This is as light as I feel it practicable to go, and I doubt whether such a form of construction could be cheapened by substituting a prestressed trussed arch. This construction gives a weight of about 660 lbs. for the tied rib including all sag rods and shoe and purlin cleats, and makes it a very favourable alternative to the other truss types illustrated in Fig. 16.

Whilst feeling that no real advantage can be gained from prestressing steel unless the structure of itself is heavy, I am much indebted to Professor Robertson for his suggestion that the arch form of rib offers possibilities; and I agree that this construction could be very usefully pursued.

In conclusion it might be of interest to know that further purlin tests have been carried out at the School of Military Engineering on $3\frac{1}{2}$ in. $\times$ $2\frac{1}{2}$ in. $\times$ $\frac{1}{4}$ in. angle purlins spaced at 4 ft. 6 in. centres and spanning 16 ft., in order to confirm that the results obtained from the earlier tests at smaller spans would hold for a range. Almost identical results were obtained, and an applied superimposed load of 30 lbs. per sq. ft. was sustained without signs of real distress.

Institution Notices and Proceedings

IMPORTANT NOTICE TO SUBSCRIBERS

Subscribers, Newsagents and Subscription Agencies at home and abroad are asked to note that as from the 1st January, 1955, copies of THE STRUCTURAL ENGINEER can only be obtained on application to the Secretary of the Institution of Structural Engineers, 11, Upper Belgrave Street, London, S.W.1.

Subscription Rate

The Annual Subscription, post free, to any part of the world is 44s. 6d.

Single Copies

The price of single copies is 3s. 6d. or 3s. 9d. if sent by post.

In order to fulfil all demands, subscribers and others are requested to state their requirements as soon as possible.

The usual rate of discount will apply.

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 27th January, 1955, at 5.55 p.m., Dr. S. B. Hamilton, M.Sc., B.Sc.(Eng.), A.R.C.S., M.I.C.E., M.I.Struct.E. (President) in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

Students

Atkinson, John Stuart, of Scunthorpe, Lincs.
Chan Siu-Pong, of Loughborough, Leics.
Hall, John Stuart, of St. Helens, Lancs.
Sharma, Vipan Kumar, of Loughborough, Leics.
Thurston, Clifford Charles, of Germiston, Transvaal, S. Africa.

Graduates

Anderson, Alexander David, B.Sc.(Civil) Edinburgh, of Edinburgh.
Banerjee, Sukumar, B.E.(Civil) Calcutta, of Stafford.
Bondale, Dattatreya Shridhar, B.E.(Civil) Bombay, of Bombay, India.
Hall, Charles Wilson, B.E.(Civil) N.Z., of Glasgow.
Hansed, Jack, B.Sc.(Civil) Birmingham, of Manchester.
Isaacs, Leslie Balfour, B.Sc.(Civil) Cape Town, of London.
Kasiraj, Ivadurai, B.E. Madras, of Cambridge, Mass., U.S.A.
Thadani, Bhagwan Nebhraj, B.E.(Civil) Poona, of Bombay, India.
Threlfall, Brian David, M.A.(Cantab.), of New Malden, Surrey.

Associate-Member

Bailey, Leonard Warner, of Leicester.

Members

Griggs, Lt.-Col. Francis Edward, M.A.(Cantab.), A.M.I.C.E., of Borough Green, Kent.
Linn, William, M.I.C.E., of Glasgow.

TRANSFERS

Students to Graduates

CANESSA, Eric Albert, of Gibraltar.
GRABHAM, Frederick Robert, of London.
MILES, Alan David, of Sidcup, Kent.
MULHALL, Kenneth, of Warrington, Lancs.

Graduates to Associate-Members

COLE, Eric Maxwell, of Newhaven, Sussex.
HAWKER, Geoffrey Fort, B.Sc.(Eng.) London, of Woodford Green, Essex.
O'CONNELL, William Noel, B.E., N.U.I., of Sutton, Surrey.
ZELMAN, Isaac Maier, B.Sc.(Tech.) Manchester, of Weybridge, Surrey.

Associate-Members to Members

HALLOWELL, Harry, of Birmingham.
HUGHES, Edward, of Bristol.
MANDER, George Charles, B.Sc.(Civil) Bristol, M.I.C.E., of Bristol.

Associate to Retired Associate

LLOYD, John Wesley, of Luton, Beds.

RE-ADMISSION

Associate-Member

ALEXANDER, Angus George Forbes, of Penhalonga, Southern Rhodesia.

OBITUARY

The Council regret to announce the deaths of Harry Hayes CLAPHAM (Member) and John HURDITCH (Associate-Member).

RESIGNATIONS

Notification was given that the Council had accepted with regret the resignations of John Alexander Ogg ALLAN, George HENDERSON, Douglas Stewart YOUNG (Members); James JACKSON, Donald Arthur STEVENS (Associate-Members); Eamon Peter FITZGERALD, K. Adolph KAYE, Arthur Hiram WAGNER (Graduates); John Thomas ANDREWS, James Bronte ARRAND (Students).

EXAMINATIONS—JULY, 1955

The Examinations of the Institution will next be held at centres in the United Kingdom and Overseas on July 12th and July 13th, 1955 (Graduateship), and July 14th and 15th (Associate-Membership).

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, March 24th, 1955

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Dr. E. H. Bateman, M.A., B.Sc., M.I.C.E., M.I.Struct.E., A.M.I.Mech.E., A.F.R.Ae.S. (Member of Council) will give a paper on "The Development of the Elastic Theory of Continuous Frames"; and Mr. R. J. Wilkins will demonstrate his visual aid apparatus for structural analysis.

Thursday, April 28th, 1955

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. G. P. Bridges, M.I.Struct.E., A.M.I.C.E., L.R.I.B.A., will give a paper on "Colliery Structures."

Thursday, May 26th, 1955

Ordinary General Meeting for the election of members.

Annual General Meeting of the Institution.

Annual General Meeting of the Voting Contributors to the Institution of Structural Engineers' Benevolent Fund.

Members wishing to bring guests to the Ordinary Meetings referred to above are requested to apply to the Secretary for tickets of admission.

Cancellation of Meeting

There will be no Ordinary Meeting on Thursday, March 10th, 1955.

SESSIONAL PROGRAMME

The Literature Committee have under consideration the selection of papers for inclusion in the Sessional Programme for 1955-56. Members who may wish to offer papers during the coming Session are invited to communicate with the Secretary.

MACLACHLAN LECTURE COMPETITION, 1955

The closing date for the receipt of entries for the next Maclachlan Lecture Competition is Thursday, March 31st, 1955. The general conditions of the competition are as follows:

1. The Institution of Structural Engineers shall institute a written lecture to be known as the Maclachlan Lecture and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Engineering as long as in every second year the subject shall be confined to steel structures. (This will be the case in 1955.)

3. Entrance into the competition for the Lecture shall be confined to Associate Members of the Institution who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning Lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award, or for any other reason, the Institution shall transfer the above sums to the Research Fund of the Institution.

PARTICULARS OF THE COMPETITION FOR 1955

1. The Maclachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1955.

2. The subject of the Lecture shall be confined to steel structures.

3. The work should be submitted as the script of a lecture which the author, if successful in the competition, will deliver before an audience in the course of about one hour. The development of mathematical

formulæ and detailed calculations should be avoided as far as possible in the text; if they are essential they should be embodied in appendices. Photographs, drawings, graphs, etc., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.

4. Six copies of each Lecture should be submitted and should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the Institution is Thursday, March 31st, 1955.

DRURY MEDAL AWARD

The fifth competition for the above award will take place in 1955. The subject is the design of a mobile crane.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary : envelopes to be marked in the top left-hand corner, " Drury Medal Award."

The closing date for the competition is October 1st, 1955.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say, primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution, or failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate in the presence of a Justice of the Peace or a Commissioner for Oaths.

JOURNAL CASES AND BINDING, 1954

A binding case can be supplied for the twelve issues of the Journal, January-December, 1954 (Volume 32), price 11s. 6d., post free. The price for binding in half-leather, inclusive of packing and postage, is 27s. per volume.

It is requested that all parcels containing Journals forwarded for binding should bear the name and address of the sender and must be despatched to reach the Institution by the 4th April, 1955.

An Index will be included in all bound volumes. Members making their own arrangements for binding may obtain a copy of the Index upon application to the Secretary.

Copies of the Index will be provided for subscribers and those on the complimentary list.

INVITATION TO SUSSEX MEMBERS

The paper on " The Design and Construction of the Nuclear Reactor Buildings at Windscale Works, Sellafield " which Mr. D. R. R. Dick gave before the Institution on November 25th is to be repeated at a meeting of the Southern Association of The Institution of Civil Engineers to be held at Brighton Technical College on Thursday, March 24th, 1955, at 6 p.m. Members of the Institution of Structural Engineers resident in Sussex who may have been unable to hear the paper in London are cordially invited to attend this meeting and will be welcome to join members of the Association for tea at 5.30 p.m. In order that catering arrangements may be made, members who intend to be present are requested to notify the Honorary Secretary Mr. I. Wilson, B.Sc., A.M.I.C.E., City Engineer & Surveyor's Office, " Greyfriars," Chichester, Sussex.

LONDON GRADUATES' AND STUDENTS' SECTION

The Annual General Meeting of the Section will be held at 11, Upper Belgrave Street, London, S.W.1, on Tuesday, March 15th, 1955, at 6 p.m., and will be followed by a film display.

Hon. Secretary : J. A. Pope, 53, Cranleigh Drive, Leigh-on-Sea, Essex.

CONFERENCE ON WELDED STRUCTURES, 1953

The papers of the above conference, which was organised by a Joint Committee of the Ministry of Works, the Institution of Civil Engineers and the Institution of Structural Engineers, have now been published by Her Majesty's Stationery Office, price 40/-. The publication includes a record of the discussions on the papers.

EXAMINATIONS

Preparation for the Examinations of the Institution by Attendance at Technical Colleges

A candidate for Graduateship or Associate-Membership may be able to attend a technical college; these notes are intended to guide him in choosing the most suitable instruction.

Preparation for the Graduateship Examination

Technical Colleges offer :

(a) Full-time courses for degrees or Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At technical colleges courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

Preparation for the Associate-Membership Examination

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering these cover only part of the requirements for the Associate-Membership Examination.

Colleges in List "A" provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

LIST "A"

Bath Technical College.
Belfast College of Technology.
Birmingham College of Technology.
Bolton Municipal Technical College.
Bradford Technical College.
Bridgend Technical College.
Chesterfield College of Technology.
Derby Technical College.
Dudley and Staffordshire Technical College.
Glasgow Royal Technical College.
City of Liverpool College of Building.
L.C.C. Brixton School of Building, S.W.4.
L.C.C. Hammersmith School of Building and Arts and Crafts, W.12.
Manchester College of Technology.
Middlesbrough, Constantine Technical College.
Nottingham and District Technical College.
Salford, Royal Technical College.
South-East London Technical College, Lewisham Way, S.E.4.
South-West Essex Technical College, Walthamstow, E.17.
Stafford, County Technical College.
Stockport College for Further Education.
Twickenham Technical College.
Willesden Technical College, N.W.10.

Colleges in List " B " provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete.

LIST " B "

Brighton Technical College.
Cardiff Technical College.
Edinburgh, Heriot-Watt College.
Huddersfield Technical College.
Leeds College of Technology.
London, Battersea Polytechnic, S.W.11.
London, Northampton Polytechnic, E.C.1.
L.C.C. Westminster Technical College, S.W.1.
Newcastle-upon-Tyne, Rutherford College of Technology.
Plymouth and Devonport Technical College.
Preston, Harris Institute.
Rotherham College of Technology.
Wigan Mining and Technical College.
Woolwich Polytechnic, S.E.18.
West Ham College of Technology.

Students are advised to take the organised courses in Structural Engineering where these are available.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Wednesday, March 30th, 1955

Joint Meeting with the Institute of Welding (Manchester Branch), College of Technology, Manchester, at 6.30 p.m. Mr. E. Lightfoot, M.Sc.(Eng.), A.M.I.C.E., A.M.I.Struct.E., will give a paper on " Dynamic Stresses in Structures."

Thursday, April 28th, 1955

At the College of Technology, Manchester, at 6.30 p.m. Annual Business Meeting, followed by a film.

Joint Hon. Secretaries : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire ; M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford, 6, Lancs.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Wednesday, March 9th, 1955

At Loughborough College of Technology, Loughborough, at 7 p.m. Dr. J. M. Harvey, M.Sc., and Mr. E. Griffin, on " The Design and Manufacture of Cold Formed Steel Sections as Structural Members."

Friday, March 25th, 1955

At the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m. Mr. V. H. Lawton, M.I.Struct.E., on " Some Notes on the Intended Revisions to B.S.449."

Friday, April 22nd, 1955

Annual General Meeting, followed by Joint Meeting with the Reinforced Concrete Association, James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m. Mr. J. A. Derrington, B.Sc., D.I.C., A.M.I.C.E., A.M.I.Struct.E., will give a paper on " Recent Developments in the Technique of Precasting Concrete Structures."

Tuesday, April 26th, 1955

At the Gas Showrooms, Nottingham, at 7 p.m. Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E., M.I.Struct.E., M.Inst.W.E., M.Am.Soc.C.E., M.Cons.E. (Hon. Librarian), on " Some Unusual Industrial Structures."

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES GRADUATES' AND STUDENTS' SECTION

The following meetings have been arranged :

Wednesday, March 30th, 1955

At the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m. Mr. C. B. Brewington, B.Sc., A.M.I.C.E. (Graduate), on " Load Bearing Walls."

To be followed by Annual General Meeting.

Hon. Secretary : A. K. A. Costain, A.M.I.C.E., 134, Witherford Way, Weoley Hill, Birmingham, 29.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, March 1st, 1955

At the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, at 6.30 p.m. Mr. M. F. Palmer, M.I.C.E., M.I.Struct.E., on " Fabrication and Erection of Steel Bridges."

Wednesday, March 2nd, 1955

The above meeting will be repeated at the Neville Hall, Westgate Road, Newcastle upon Tyne, at 6.30 p.m.

Tuesday, March 16th, 1955

At the Neville Hall, Westgate Road, Newcastle upon Tyne, at 6.30 p.m.

Ladies' Guest Night.

Wednesday, March 17th, 1955

At the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, at 6.30 p.m.

Ladies' Guest Night.

Tuesday, April 5th, 1955

Annual General Meeting at the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, at 6.30 p.m., when Mr. D. J. Leggot, A.M.I.C.E., A.M.I.Struct.E., will give a paper on " Some Aspects of Industrial Building Design."

Hon. Secretary : Captain O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, March 1st, 1955

At the College of Technology, Belfast, at 6.45 p.m. Film Evening.

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., " Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Tuesday, March 29th, 1955

At the Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, at 7 p.m. Mr. D. R. Dick, B.Sc., M.I.C.E., on " Design and Construction of Nuclear Reactor Building, Windscale Works, Sellafield."

Tuesday, April 26th, 1955

At the Cadoro Restaurant, Union Street, Glasgow, at 6 p.m. Annual General Meeting.

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., " Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, March 18th, 1955

At Plymouth, at 7 p.m. Mr. D. R. R. Dick, B.Sc., M.I.C.E., on " Design and Construction of the Nuclear Reactor Building at Windscale, Cumberland." (Mr. S. G. Silhan, A.M.I.Struct.E., will read the paper on behalf of the Author.)

Friday, May 20th, 1955

Annual General Meeting, at the Duke of Cornwall Hotel, Plymouth, 7 p.m.

Joint Hon. Secretaries : E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay; C. J. Woodrow, " Elstow," Hartley Park Villas, Tavistock Road, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Tuesday, March 15th, 1955

At the South Wales Institute of Engineers, Park Place, Cardiff, at 6.30 p.m. Mr. A. C. Vivian, B.Sc., A.C.G.I., M.I.C.E., M.I.Struct.E., on " Kathodic Protection of Steel Structures."

Wednesday, March 30th, 1955

At the Mackworth Hotel, Swansea, at 6.30 p.m. Major A. F. Allen, M.I.C.E., on " Description of Neat By-Pass Constructional Work." (Illustrated with lantern slides.)

Friday, April 22nd, 1955

Annual Dinner at Porthcawl.

Tuesday, May 3rd, 1955

Annual General Meeting, at the Offices of the Steel Company of Wales, Port Talbot, at 6.30 p.m.

Hon. Secretary : K. J. Stewart, A.M.I.C.E., A.M.I.Struct.E., 15, Glanmor Road, Swansea.

WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Thursday, March 3rd, 1955

Combined Meeting with the Institution of Civil Engineers, in the Reception Room, University of Bristol, at 6 p.m., when Mr. W. J. Sivewright, M.A., A.M.I.C.E., and Mr. S. P. Whittington, B.Sc., A.M.I.C.E., will give a paper on " Reconstruction of Ground Floor of Cold Store, Avonmouth Docks."

Friday, April 1st, 1955

At the University of Bristol Geology Lecture Theatre at 6 p.m. Annual General Meeting, followed by a film show.

Hon. Secretary : E. Hughes, M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, March 16th, 1955

To be arranged.

Friday, March 18th, 1955

Annual Dinner and Dance, at the Parkway Hotel, Bramhope, Leeds, at 7 p.m.

Wednesday, April 20th, 1955

At the Great Northern Hotel, Leeds, at 6.30 p.m. Annual General Meeting, followed by a paper to be arranged.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., P.O. Box No. 3306, Johannesburg, South Africa.

During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone : 34-1111, Ext. 257.

Natal Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co. Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

Colliery Structures*

By George P. Bridges, M.I.Struct.E., A.M.I.C.E.

Fig. 1.—General view of typical Colliery surface development

Synopsis

A BRIEF description is given of the organisation of the coal industry under the National Coal Board development of new and existing mines, foundations with special reference to building on colliery refuse heaps and fillings, architectural design of colliery structures and buildings, headframes and heapstead buildings for drum winders, friction winders, mine ventilation, airlocks. The Koepe system of winding, ground type and overhead winders, colliery buildings, storage bunkers, choice of structural and cladding materials.

Introduction

The National Coal Board is the administrative body of the coal industry in Great Britain. The organisation is divided into nine geographical Divisions, each having its Divisional Board. There are, altogether approximately 900 pits which are grouped into area organisations, the areas varying in size from six to twenty pits and each area is under the charge of an Area General Manager. The size and output of each pit vary considerably, the smaller pits having an output of 200 to 300 tons per day, and the larger pits 5,000 to 8,000 tons per day. It will be obvious from these brief statistics that the extent and capacity of the equipment will vary as much as the individual pit output. As a means of increasing production, the National Coal Board has planned many new collieries, and in other cases completely re-organised existing collieries by introducing new machinery and equipment. This re-equipment and development work involves the construction of many buildings and structures of all kinds.

*Paper to be read before a meeting of the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 28th April, 1955, at 6 p.m.

Fig. 1 shows a general view of a typical colliery reconstruction. In this case the equipment is designed to increase the output by about 400 per cent, and the colliery has produced coal regularly throughout the whole of the construction period. The whole of the output of coal is brought up the main shaft, whilst another is used for winding dirt, materials and men, and a further shaft for pumping, ventilation and emergency winding. The main shaft where the coal is wound has a larger head frame than the others, and can be easily recognised in the illustration. The coal is brought to the pit bottom in mine cars. In some installations it is tipped at this point into skips which are wound and discharged at the surface on to collecting conveyors. In other cases the mine cars are wound in cages and tipped at the surface. The method employed depends to some extent on local circumstances, but also on the opinion of the Mining Engineer concerned.

The collecting conveyor belts at the surface are usually inclined to deliver the coal to run-of-mine storage bunkers, which act as a reservoir in case of breakdowns, and from there to the Coal Preparation Plant. In this plant the coal is cleaned and screened into varying sizes, and delivered either into railway wagons, or to further belt conveyors to landsales bunkers, power stations, etc. The dirt or refuse may be wound in tubs separately, or taken out of the coal in the preparation plant from where it is conveyed to the tipping ground or alternatively treated for stowing in the pit.

Foundations

In some cases it is necessary to put down trial loads to ascertain the bearing value of the ground without serious settlement ; these tests however are of limited value and the results can be misleading, because of the small test load compared with the actual load to be carried.

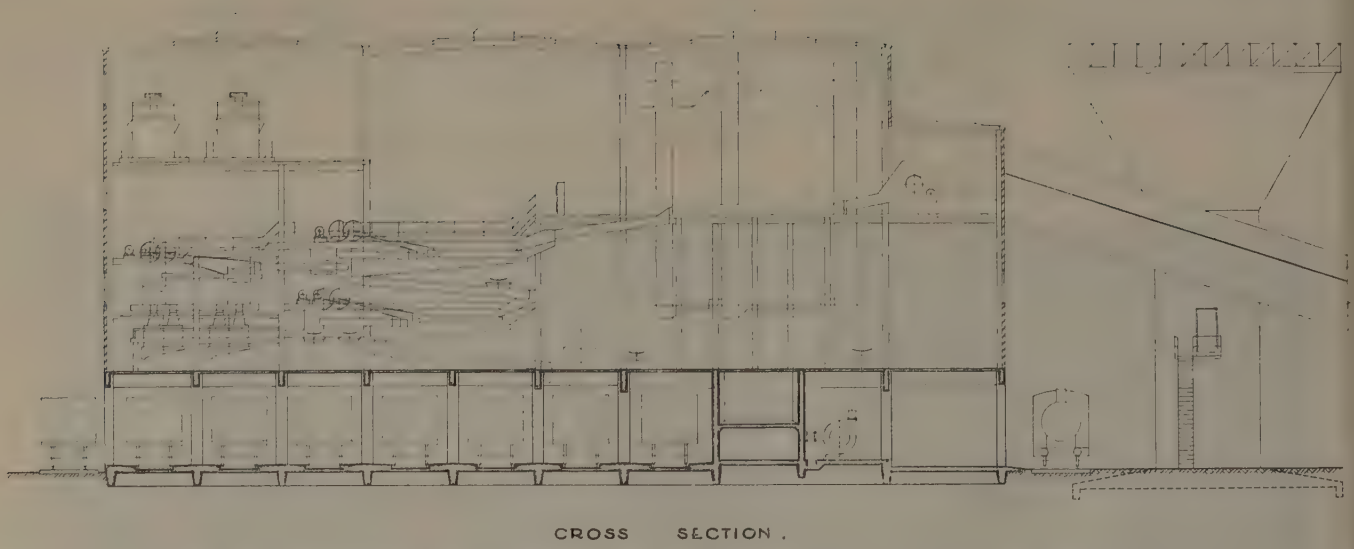


Fig. 2.- Cross section of foundation raft and substructure for Coal Preparation Plant

It is impossible to lay down any rules to arrive at the bearing value of the ground, but a few general principles can be stated. Take the case of a comparatively light structure of low height and covering a large area, such as colliery screening plant. This could be built on soft ground or filling using shallow raft or strip foundations, but comparing this with a high heavy structure such as tower, bunker or chimney, where wind pressure is a factor, the foundations would either have to be taken down to the depth of a hard clay or rock stratum, or alternatively piled. Whereas in the first case the pressure on the ground would be more or less uniform over the whole area, in the second case the pressure at the edge of the foundation would be increased owing to the wind overturning moments. Similar conditions may be set up for heavy vibrating machines. The cost will have to be considered in solving every foundation problem.

For some structures, for economic reasons, a limited amount of settlement can be allowed. If such settlement is uniform no damage will occur, but trouble will arise if the settlement is uneven.

Many structures on collieries are situated on refuse tips or on ground which consists of filled colliery refuse, sometimes to quite a considerable depth. In such cases, the foundations will have to be either rafts or piles. It is usually cheaper to use rafts, and in fact there are many cases where piling would be impossible. Raft foundations should be designed to give a uniform pressure over the whole area, but this is not always possible to achieve, particularly when bunkers or tanks are incorporated in the structure and various compartments are continually being emptied or filled. This condition often occurs in the case of Coal Preparation and Screening Plants. In the case of reinforced concrete structures, advantage can be taken of the reinforcement tying the foundation to the structure, and thus forming a monolithic raft of tremendous strength which assists in overcoming the variation in loading referred to above. Fig. 2 shows a typical cross section of a Coal Preparation Plant, showing how the strength of the raft is increased in this way. Very often concrete pavings are required at the ground level (in this case under the railway sidings), and these paving slabs can be incorporated in the raft design and will result in the utmost economy of materials.

Aesthetics

Good planning must give first consideration to the operational requirements, but a good deal can be done in the orderly arrangement of the various units and structures and careful layout of the sidings and roads to improve the general appearance and efficiency of the surface plant. Ample space should be allowed in the layout for any possible future additions or extensions, and the introduction of grass plots, trees and shrubs will enhance the general appearance of the pit, providing they are not too formal or ornamental.

The headframes and buildings of most collieries form prominent features in the landscape, and these structures should have a good architectural appearance. The successful design depends upon careful proportioning of the principal masses, the orderly arrangement of structural members for use as features of interest, and the good proportioning and arrangement of the window, door and other openings. These features can be arranged to obtain bold shadows in order to create contrast.

Added ornament in the form of pilasters, cornices, panelling and other classical detail are entirely out of place in colliery structures. Apart from the added cost of such features, their use generally results in bad design. On most structures such features as stairs, landings, conveyors, elevators, brackets, piping, lifting beams, etc. are conspicuous features, and it is just as necessary to consider their design and arrangement as it is important to consider the design of the buildings themselves. Good architectural design should not involve any increase in capital cost, but should result in economy.

There are fashion phases in architectural treatment as well as in ladies' clothes, and some engineers must be in the fashion. This often results in the unsuitable application of architectural styles with consequent waste and bad building.

At the present time glass walls and dragon tooth windows are very fashionable, although they are often ill-applied. Good lighting is very essential for colliery buildings, but glass is not an ideal material for wall cladding as it has no structural value, a very low degree of insulation and is a further maintenance liability. What is more important is that its cost, as a cladding material, is at least twice that of $4\frac{1}{2}$ in. brickwork.

Fig. 3.—Illustrations of Head Frame and Airlock over Upcast Shaft.

Ventilation Effects

The normal method of ventilation is by a suction fan situated as near as possible to the mine shaft. There are normally two shafts, one downcast which is open at the surface, and the other the upcast which is connected to the fan by means of a drift and sealed at the surface. Ventilation is effected by drawing the air into the downcast shaft through the workings, up the upcast shaft and discharging on the atmosphere side of the fan. The resistance built up in the passage of the air through the system varies considerably, and considerable effort is made to streamline the shafts and workings to reduce the fan pressure to a minimum.

On the Continent pressures as much as 24 in. water gauge are common, but in Britain the fan pressures do not as a rule exceed 8 in. to 10 in. water gauge. The higher fan pressures increase the size and power of the fan, and also the horse-power consumed, and consequently high pressures mean higher power costs. To prevent short circuiting of the air, the upcast shaft must be completely sealed off at all times from the atmosphere by means of airlocks and housings designed to take the maximum fan pressure; alternatively, steel casings around the shaft guide framing are used.

On some collieries force fans are used as an alternative to suction fans; in such installations the directions of the air currents are reversed, otherwise the problems presented are the same.

To seal the upcast shaft it is very often economic practice to erect a tower to enclose completely the shaft and guide framing up to the overwind level, and also to enclose completely the heapstead buildings for housing the mine car tracks and equipment. (See Fig. 3). This arrangement necessitates the provision of separate airlocks with mechanically operated doors for the mine cars, and also revolving doors or special airlock chambers for the men. The airlock tower can be used as a headframe to support the pulleys, and usually has ample strength and stability to take the rope pulls.

The more up to date method of enclosing the upcast shaft is by the provision of specially designed steel aprons fitted with pneumatically operated doors to receive the cage both at the discharge level and in the overwind position. Difficulties arise in the design of this type of airlock when dealing with cage or skip changing, and also for the entrance of the men into the pit.

Headframes

The headframe over the pit shaft is usually a conspicuous structure on a colliery because of its height which may be up to about 200 ft. above ground level, and to its frame like structure. In Britain normally drum type winders were used almost exclusively until recent introduction of a Koepe winder. The ropes in the case of the drum winder are secured to the winding drum at one end, and to the pit cage at the other. There are usually two ropes fitted to the drum, one rope winding on and one off, with a consequent raising and lowering of the cages in order to balance the pull. The ropes are coiled on to the drum as the cage rises to the surface, and off the drum as the cage is lowered. Steam driven winders were most commonly used, and there are many still in use, but in more recent years most new winders are electrically driven.

The policy of the National Coal Board has been to increase the output of each individual pit as much as possible, and consequently larger capacity mine cars and higher capacity skips have come into use. In addition shaft depths have increased in order to exploit the deeper seams of coal, and these developments have resulted in larger pay loads and faster winding times, which in turn have necessitated increased rope length and diameter. The longer, bigger and heavier ropes have created difficulties in the design of the winding drums, and this has resulted in the adoption of a friction or Koepe type winder as an alternative to the drum type.

Koepe Winders

The Koepe system bears the name of its inventor, a German Engineer. This system is referred to as a friction winding system, particularly when referring to multi rope wheels, and it came into use on the Continent late in the last century. It is only in recent years that it has been used in Britain, but owing to the increased depth of seams and increased pay loads, its use is now becoming standard practice in Britain.

The principle on which the system is based is simple. It consists of a continuous rope to which are attached two cages or skips, arranged in such a way that one cage is level with the loading point at the bottom of the shaft at the same time as the other cage is level with the discharge point at the surface. The rope passes over a Koepe (driving) wheel, which is electrically driven, and the periphery of which is fitted with a friction material grooved to take a single rope. The arc of contact which the rope makes with the wheel produces sufficient force through friction to lift the pay-load. An additional rope is fixed to the underside of the two cages and is called tail or balance rope, which hangs freely in the shaft and which is looped below the lowest winding position of the cages, so that the weight of the rope and the cages is completely balanced in all positions. (See Fig. 4).

The above diagram shows the driving wheel mounted at the top of the tower, but as an alternative the drive can be situated at the ground level, and the rope passed over head pulleys similar to those used for drum winders.

The advantages of the Koepe system of winding are that it facilitates larger ropes, and can, therefore, be used for mining at greater depths and with heavier pay-loads. The speed of winding can also be increased and the horse-power reduced.

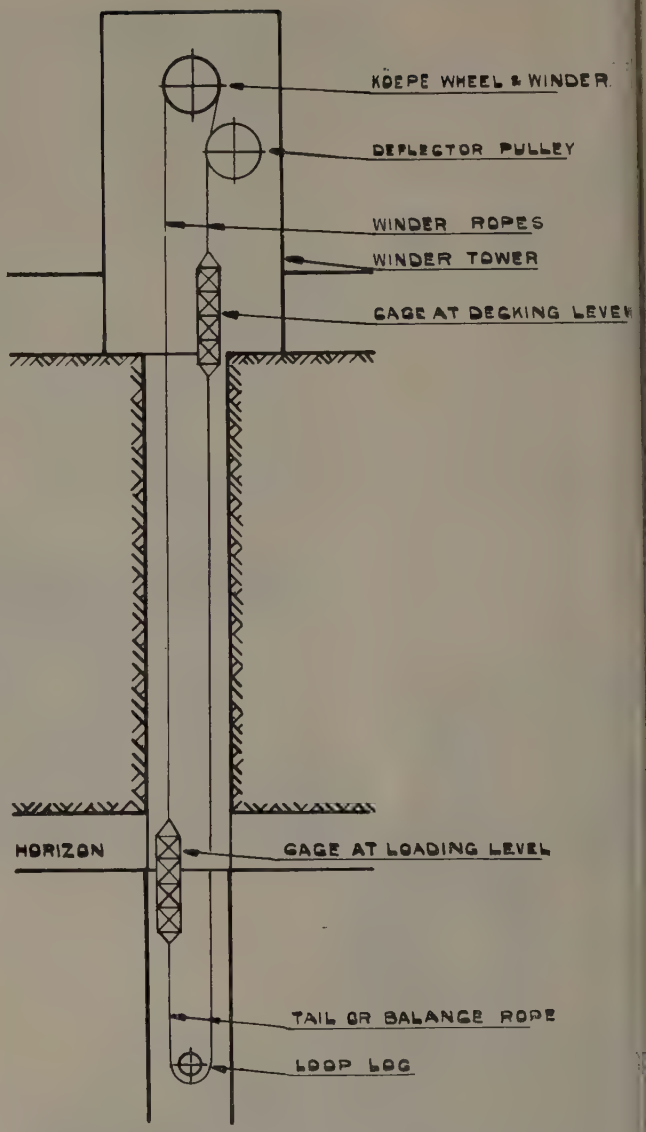


Fig. 4.—Diagram of Koepe Winder installation

Originally all Koepe winding installations used a single rope, but recently multi-rope suspension systems employing two or four ropes have come into use. The multi-rope system allows smaller diameter Koepe wheels to be used and consequently smaller buildings to house them ; this also simplifies rope changing and eliminates defector pulleys and reduces the height of the headframe. Fig. 6 shows arrangements of typical Koepe installations.

(*a*) Single Koepe wheel (2 cages) with a single stay headframe and ground mounted winder.

(*b*) Double Koepe wheel (4 cages) with double stay headframe and ground mounted winder.

(*c*) Single Koepe wheel (2 cages) with multi rope tower mounted winder.

(*d*) Double Koepe wheel (4 cages) with tower mounted winder.

Headframe Design

The conventional type of headframe over the upcast shaft combines the headframe tower and the heapstead area in one building as shown in Fig. 3 with airlocks on both sides of the pit for the entrance and exit of men and mine cars. It is necessary to fit special

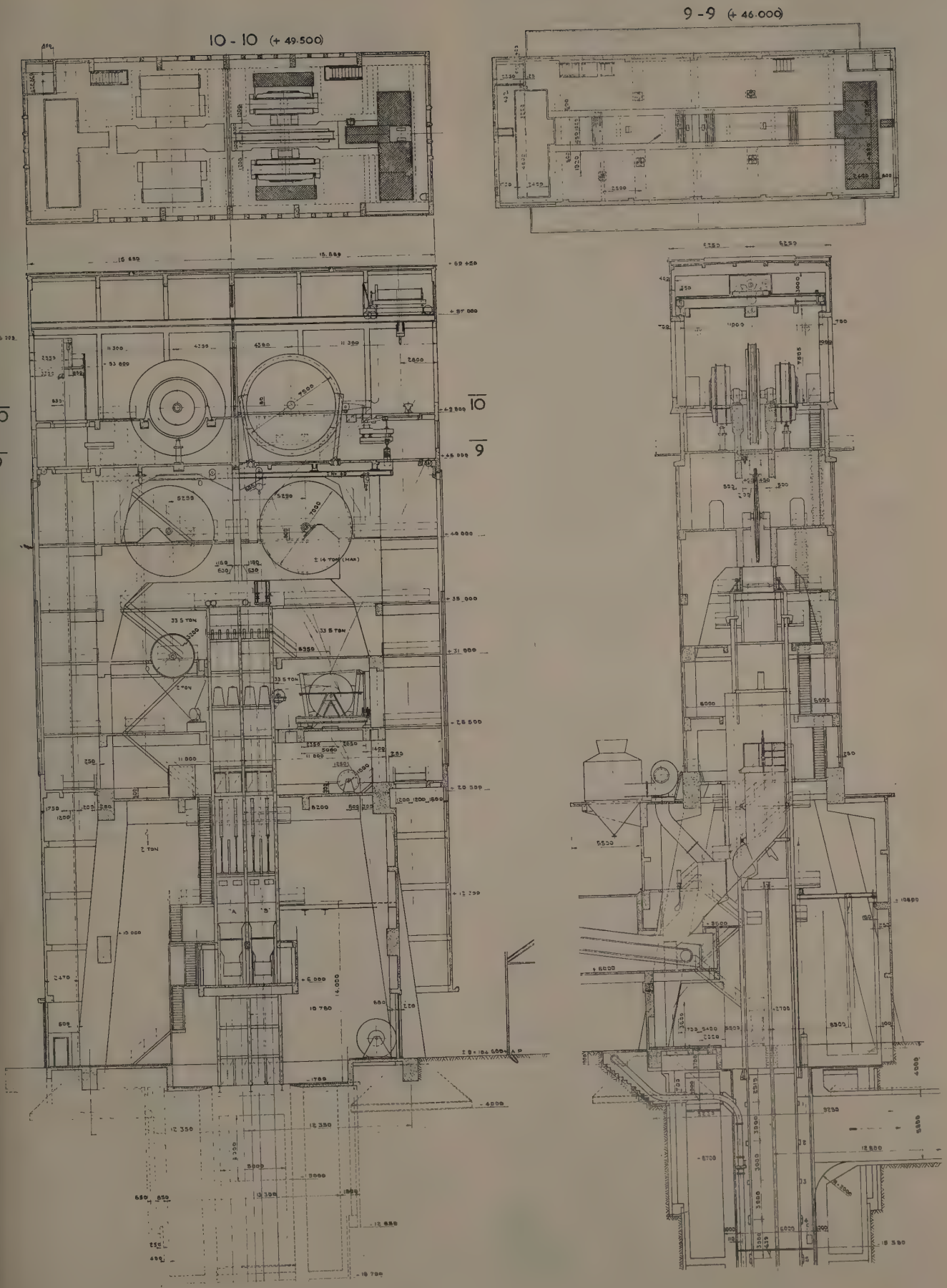


Fig. 5.—Arrangement of Koepe Winder, Tower Type

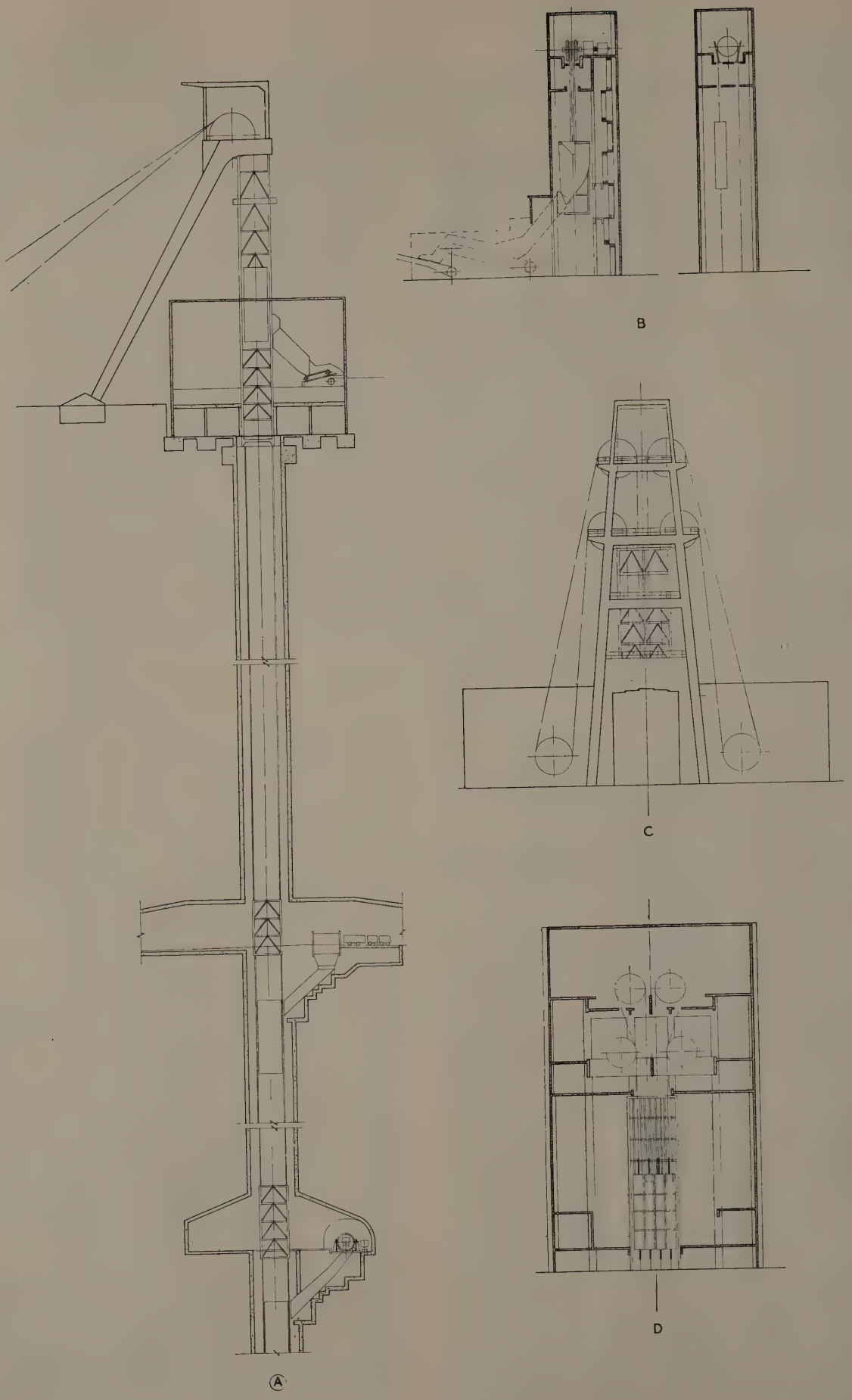


Fig. 6.—Diagrammatic view of four types of Koepe installations

doors to these airlocks to make sure that the air seal remains unbroken throughout all the operations. Large airlocks are also provided, usually at the ground level, for changing cages and bringing bulk materials into the pit. The whole of the structure must be designed to withstand the fan air pressure. For this reason airlocks are usually constructed in reinforced concrete. The airlock tower accommodates the guide framing or rope guides, according to which is employed, and these are supported on the members spanning across the top of the tower. The sealed portion of the tower is taken up to the guide support platform where the detaching mechanism is provided to allow the cage to run on in case of overwind. Provision is also made to support the cage by means of special catches to prevent this falling back into the shaft. Access stairs are provided to the cage level in its overwind position, and some Authorities insist that accommodation should be arranged at this level to take stretcher cases out of the cage in case of accidental injury to men.

In more recent designs sheet steel casings are provided at both the decking and overwind levels, and special provision made in the design of the cages to seal off completely the shaft when the cage is being decked or an overwind has occurred. Special steel flap gates are also provided sometimes to seal off the shaft during cage changing operations, or when repairs are being carried out to the equipment. In this type of design the airlock buildings and doors can be dispensed with, but sealing doors are necessary at each decking level on the guide framing. Although this works quite well on single deck schemes, it becomes very complicated where simultaneous decking at more than one level is used.

As an alternative to airlocks for men, it is quite common practice on more modern designs to use revolving doors for the men's entrance and exits. By this means men can enter the heapstead in a continuous stream instead of in batches, as is necessary in the case of airlocks.

The downcast shaft is usually open to the atmosphere and is used as an intake for the fan ventilation. For this reason a framed and braced structure constructed in either steel or reinforced concrete is normally used. This frame must be designed to carry all the loads from the pit equipment, together with the pulls from the winding ropes. The principal loads to be carried are the guides and guide tension weights, (there are usually about six of these in a two-cage shaft), the weight of the cages and suspension gear, together with pay loads, pulleys, etc. The load which is applied by the winding ropes is usually taken as the breaking strain of the rope, so that the headframe will be structurally sound to withstand the maximum load that it could possibly be called on to take. It is important that all members of a headframe of this type should be designed to withstand shock and vibration which is being applied to it constantly at every wind. On each occasion when the winder is started, there is always a heavy snatch on the rope which causes vibration on the frame, and for this reason all structural members must have a low stiffness ratio. It will be obvious that reinforced concrete structures have a big advantage in overcoming these forces, because of the natural stiffness of the members, and particularly the continuity at the connections. Reinforced concrete often presents some difficult site constructional problems, and because of this, in many cases the headframes have been designed as steel frames and encased in concrete after erection. Another advantage of reinforced concrete structures is that they are better able to resist corrosion.

The above remarks apply principally to drum winding installations. When Koepe winding is used the problems to be solved in the design of the headframe are much more involved. The problems can be divided into two sections :

(*a*) frames for ground mounted winders.
(*b*) frames for overhead mounted winders.

See Fig. 6.

Variations to these installations which affect the design are that in either case they may be a single winder when there are two cages or skips in the shaft, or a double winder when there are four cages or skips in the shaft. Fig. 6a shows a simple single ground type winder installed over a downcast shaft. The duty of the frame in this case is very similar to the frame used for the drum winder, but as it is necessary to allow greater overwind distances and usually larger and heavier cages and equipment are employed in Koepe installations, the height of the frame is greater than is usually necessary from the drum winder. In the case of a larger installation with four cages or skips in a shaft, a double winding equipment is required, and an installation of this type is illustrated in Fig. 6c. The problems involved are very similar to those in the single winder frame, but the frame is usually larger and has a wider spread at the ground level to withstand the rope pulls, which in this case come from two opposite directions. The Koepe wheels are accommodated in the winder houses, together with the winder mechanism, and the framing supports the carrying pulleys at the top and the deflector pulleys immediately below them. In Koepe installations, although the rope guides are sometimes used, the majority of cases have fixed guides. There are usually two guides to each cage or skip, and these are fixed to buntings attached to the shaft lining, which are usually in reinforced concrete. The guide framing above ground level is supported on the shaft collar, and consequently the frames in such cases do not have to carry the guides or guide tension weights, so that the loads coming on the frame are usually the dead weight of the cages and suspension gear, the pulleys and pedestals and the winder pull. As in Koepe installations, the cages are attached to a continuous rope. There is no detaching mechanism between the rope and the cage, and in the case of an overwind, should the motor continue running after the cage is stationary, the rope will slip on the wheel. For this reason it is not necessary to design the headframe to withstand the breaking strain of the rope. The winder houses in these installations should be positioned as near as possible to the shaft. This increases the angle of winder rope to the horizontal but reduces the overturning pull on the frame. In doing this it is necessary for the rope to pass through the roof of the winder house, and this does create weathering problems. It is nearly always necessary to provide a heapstead building, and it is far better to incorporate this building in the design of the headframe. This allows for better stiffening of the members, and results in overall economy. On both the drum and the Koepe type of tower, provision is always necessary for changing the pulleys. As this is rather a hazardous operation, special provision is made on all modern installations by providing adequate lifting beams to facilitate wheel changing in a minimum of time. Clearances have to be carefully worked out to allow pulleys to pass between the members, and lifting beams provided for lowering the pulleys to the ground level.

In conventional installations a winder man is accommodated in a special cabin inside the winder house to operate the winding mechanism, but in more recent installations automatic winding is being employed, and manual operation is only then required in case of emergency, or for maintenance operations. For this reason it is better to accommodate the winder cabin on the shaft side of the winder house, so that the winder man has a view of the shaft top as well as of the winder. This arrangement is not being accepted very readily at the moment, but will be used in many future installations.

The driving motors can be for alternating or direct current. If reducing gears are necessary, the space required to accommodate this type of drive can be considerable. With direct current, the motors can be direct coupled to the driving wheels without introducing reduction gears, as a result of which the space required in the Winder House is very much smaller. Against this, space must be provided to accommodate the motor generator sets, and these should be situated as near the motors as possible. Expert opinion is very divided on which of these two alternatives is better, because apart from the difficulty in assessing the economics of an installation, some engineers prefer direct current because of the simplification and ease of the motor controls. From the structural engineer's point of view, the design problems are very much the same in both, but the dimensional size is reduced and the structural cost very much lower when direct coupled motors with D.C. are used.

As previously stated, the coal may be wound in steel skips or in steel mine cars. In the first case the coal is brought in mine cars to the pit bottom where they pass through a tippler and discharge into the skip. On reaching the surface the coal is discharged from the skip direct on to belt conveyors, which take it to the coal preparation plants. The skips are usually 10 to 12 tons capacity, and are designed with doors for automatic operation. Small hoppers are provided as a reservoir between the shaft and the collecting conveyors, designed to hold about three-quarters to one skip load. Some skips are also provided with special platforms which can be brought into use for man riding, but these are rather the exception than the rule. The National Coal Board have draft regulations which have been approved by the Ministry of Fuel and Power for the design of skip winding installations, and these regulations specify a minimum overwind run for skips of 15 ft. If the installation is designed for men riding, the overwind distance must be increased to 30 ft., and provision made for taking the men off at the overwind position. These provisions increase the height and also the cost of headframes considerably when designed for winding men.

As an alternative to skip winding installations, the mine cars are loaded into cages at the pit bottom, and pushed out on to the mine car tracks at decking level. The full cars then circulate through the decking system to the tippler where they are discharged on to collecting conveyors en route to the coal preparation plant. Empty mine cars leave the tipplers and circulate back to the shaft for loading into the cages as the fulls are unloaded. Mine car capacity has increased considerably in recent years, and it is very common to find modern cars of up to 3 tons capacity ; in fact on the Continent some of these cars go up to 14 tons capacity, so that they can approach the size of a pre-war railway wagon. It will be realised from this that the handling equipment for mine cars has become a very important factor in designing colliery installations, because the cannot be moved manually. They must be controlled by mechanical equipment throughout the whole of the circuits. In order to keep the area of the heapstead buildings to a minimum, traversing gear is almost universally used, and all operations on the surface are carried out by special compressed air or electrical gear. In most installations the cages are designed with three or four decks, so that the cage must be stopped three or four times at each decking period. This tends to increase the winding times, and for this reason skip winding has a big advantage.

Another important consideration in headframe design is rope changing. Under the Coal Mines' Act winding rope must be changed after specified periods of use irrespective of their condition. In most drum winding installations a $1\frac{3}{4}$ in. diameter rope was one of the largest sizes used, but in all modern Koepe winding systems the ropes may be up to 3 in. diameter, and when we consider that the depth of the modern shaft is about a 1,000 yards, it will be clear that the problem of rope changing, particularly from the weight handling point of view, is a very important one. In the case of multi-rope suspension schemes, the rope diameters are much smaller and the rope changing problems are made very much easier to cope with, but in either case special provision must be made to accommodate special rope changing winches, and clearances allowed in the headframes and guide framing to facilitate the introduction of new ropes. As a result of later improvements transportable winches are now coming into use, and for reasons of economy are bound to become more favoured, so that all installations should be designed to allow facilities for bringing these transportable winches as near to the shaft as possible.

Provision must be made on all installations to give easy access to all points where supervision and manual work have to be carried out, particularly during maintenance and breakdown periods. The most important point to be borne in mind is that the time taken to carry out repairs or maintenance jobs must be cut down to a minimum, so as to avoid losing production ; easy access, both for men, materials and supervision is of great importance. A lift, both for comparatively light spares up to about a ton in weight and for men from the ground level to the motor house in the case of a tower mounted winder, is essential ; also it is quite common today to install a light passenger lift on open headframes. Provision must be very carefully planned for lifting spare parts from the ground into the motor house and to floors at other levels. This is not always easy, particularly when the space at the ground level is congested, as in the case of a mine car installation.

It is usual to provide a sinker frame in the first instance for the sinking operations on a new shaft, and to build the permanent headframe round the sinker frame, either during sinking operations or as soon after as possible. In the past very little consideration has been given to the design of the permanent frame when putting up the sinker frames, and this has caused difficulty, both to the designer and the erector, much of the difficulty could be overcome if more thought were given to the design of the sinker frame to make sure that this will allow plenty of room for the construction of the permanent frame. It has been found possible to use the permanent guide framing for sinking purposes. To do this a temporary steel structure has to be provided to take the sinker winder pull, but this arrangement can result in a considerable saving of time and money.

Buildings and Storage Bunkers

On a modern colliery there are many buildings required to house the equipment and to provide the necessary working facilities. Principally, these are winder houses, heapstead buildings, workshops, power houses, locomotive sheds and repair shops, stores for materials such as timber, lime, cement and explosives, offices, canteens and bath houses. Buildings are also required for housing the coal preparation and distribution plant, which under modern conditions is usually the largest building on the surface. Special requirements are called for in the design of each of these buildings, in order to provide the essential facilities. Structurally the design problems are very similar to the design of any normal group of buildings, with one or two exceptions. Any buildings which house heavy vibrating machinery must be given special treatment to withstand the vibration, and in some cases such as in compressor houses, efficient insulation must be provided in the foundations. As a general principle it is more important to provide strong bulky sections and rigid connections.

Every effort should be made in the design of buildings to reduce the amount of maintenance required to a minimum. Exposed steelwork which has to be painted should be avoided as far as possible, and brickwork is preferred to concrete for the walls of most buildings.

Storage bunkers are required between the pithead and the delivery points in order to provide reservoirs and avoid losing production should a breakdown occur ; they also provide a balanced or continuous feed to the coal preparation plant, and serve to store separate sizes for loading into trucks or road vehicles, or for drainage purposes.

Coal is a friable material and storage bunkers should be designed to reduce breakage to a minimum. This could be done by the introduction of spiral cataract anti-breakage chutes or by adopting anti-breakage shapes. The problem has been dealt with very fully in another Paper by the Author.

Materials and Construction

The two alternative structural materials are steel and reinforced concrete. Steel is easily fabricated and erected and can be designed to withstand any system of pressure or forces. It is light in weight, and consequently requires a minimum of supports and foundations. Its chief disadvantage is its liability to corrosion, and for this reason it is better protected from exposure to the atmosphere wherever possible. Reinforced concrete has most of the structural characteristics of steel, and in addition it has a much higher resistance to corrosion, and consequently requires less maintenance. Brickwork is the most satisfactory form of cladding for buildings. It has a high resistance to corrosion and can be used to provide any necessary degree of insulation. For economy $4\frac{1}{2}$ in. thick brick panels can be used for most colliery structures. These are usually housed or attached to steel framing, and care is necessary in securing the panelling to the framing. One disadvantage of $4\frac{1}{2}$ in. brickwork is that clay bricks are not uniform in width, and it is difficult to obtain a uniform presentable surface on both sides of a $4\frac{1}{2}$ in. wall. For this reason concrete bricks have come into use, but they are not equal to clay bricks as a weather resisting cladding. $4\frac{1}{2}$ in. brickwork at the present time is more costly than some other cladding materials, but when maintenance and durability are taken into account, it is one of the best cladding materials.

In choosing materials for colliery buildings the time factor is important. There is at the present time a serious shortage of both skilled and unskilled workmen. This condition tends to slow down the speed of the work on the site, and results in higher costs. It is advisable, therefore, to use prefabricated materials wherever possible, and to adopt as many different structural materials as possible in order to spread the available labour over the site. For some it is advisable to use prefabricated steel in preference to reinforced concrete, and in any case it is unwise to concentrate on one structural material on any one site.

For headframes reinforced concrete is preferable to steel on account of its corrosion resisting properties, but it has the disadvantage that it absorbs a good deal of site labour, and as a rule takes longer to erect. In many cases new headframes have to be erected over existing shafts or over sinking frames, and steel presents fewer problems than reinforced concrete in such cases.

No hard and fast rules can, however, be stated because the local conditions and the availability of materials vary so much between one site and another. Each case therefore, must be considered individually, and the most economic solution chosen.

It is usual in new sinkings to erect a temporary headframe especially for shaft sinking. Under modern conditions these frames are just as expensive and heavy as many permanent headframes. Efforts should be made, therefore, to design the permanent headframes in such a way that the lower portion can be used for sinking purposes, and the erection continued on completion of the sinking. This idea has been adopted on some collieries, particularly on the Continent, with success, and it would be a great advantage if this suggestion were adopted more extensively.

Acknowledgment

All the work illustrated in the paper is taken from examples designed and constructed by Messrs. Simon-Carves Ltd. The Author wishes to record his appreciation and thanks for permission to use this material, and also to colleagues who have assisted in the preparation of this paper.

A Two-Dimensional Investigation of the End Anchorages of Post-Tensioned Concrete Beams

By S. P. Christodoulides, B.Sc.(Eng.), A.M.I.C.E., Diploma of Physics-Mathematics (Athens)

Summary

A two dimensional photoelastic investigation has been carried out to explore the distribution of stress in the end block of a rectangular beam post-tensioned by two cables with anchorages symmetrically arranged about the centre line of the end section.

This work was designed as a preliminary investigation to the solution of the complete three dimensional problem which is now in progress, and some deviations from the practical form of post-tensioned concrete units were considered permissible. Thus, the effect of the holes which are normally cast in the concrete to accommodate the cables was ignored.

By using standard photoelastic techniques, the principal stresses and the normal and shear stresses acting on transverse and longitudinal sections of the beam were evaluated. From the values obtained it was established that tensile stresses develop under the two prestressing loads, of such magnitude as to require steel reinforcement.

The distribution of principal stresses is recorded, as it is considered that failure will be governed by the maximum principal tension. The normal and shear stresses are given for comparison with the assumptions or results of previous investigators. Values of the invariant :—

Θ = the sum of the principal stresses = the sum of normal stresses

are calculated by two different methods and are compared to show the accuracy of the work.

In the theoretical treatment, the generalised plane stress solution was obtained for an infinite half-plane subjected to two constant pressure strips on its edge, which was considered to represent the loaded end section of the post-tensioned concrete beam under the two cable loads, with its longitudinal boundary planes vanishing to infinity. This simplification yielded an easy mathematical solution of the problem and proved to give good qualitative agreement with the results obtained photoelastically away from the longitudinal boundaries.

Introduction

In the design of post-tensioned concrete units, with its increasing use in important structures, the designer needs to know the distribution of stress in the end blocks. High concentrations of stresses occur around the anchorages of the cables and the concrete may fail if adequate steel reinforcement is not supplied at the over-stressed regions. Such failures have come to the author's knowledge, and have appeared as horizontal cracks developed during the post-tensioning stage.

* The index numbers refer to the items in the list of references at the end.

Professor G. Magnel*[1] has assumed that the distribution of compressive and shearing stresses along longitudinal lines in post-tensioned beams is, near the end anchorages, of the form of a "cubic parabola." In making this general assumption Magnel no doubt relied on his long and successful experience in prestressed concrete work, and a limited agreement was obtained with the results of photoelastic tests carried out in Holland by Tésar[2]. It is, however, no more than an assumption.

Y. Guyon [3] has treated the subject mathematically and used Fourier Series to obtain expressions for the stresses in a rectangular prism subjected to loads applied on one face. Guyon worked first on an infinite half-plane, and then to facilitate the introduction of the longitudinal boundaries of the prism, had to use assumptions which, in the author's opinion, distorted the solution obtained.

Both these two investigators dealt with a two-dimensional problem, and ignored the effect of the cable-ducts.

In the photoelastic investigation carried out by the author the important curves of elasticity i.e. the "Isoclinics" and "Isochromatics" were obtained experimentally on a photoelastic bench by using a model of suitable material.

An isoclinic[4] is the locus of all points in a stressed material at which the principal stresses are parallel to a certain direction (Fig. 4).

An isochromatic curve is the locus of all points in a stressed material at which the algebraic difference of the Principal Stresses is equal to a certain constant (Fig. 2). If both these curves are determined, the directions and differences of principal stresses are known, and therefore the maximum shear may be directly calculated. For if P and Q are the principal

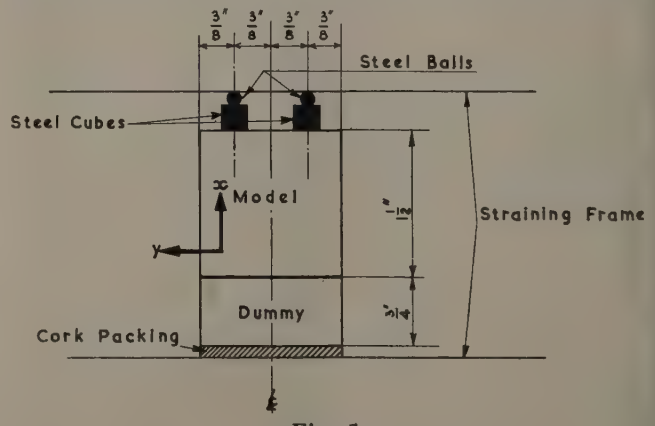


Fig. 1

stresses at a point, the maximum shear in the plane of the principal stresses is given by $\frac{1}{2}(P—Q)$ and it acts

across the plane bisecting the angle of principal planes (Table 1).

From the isoclinics, by drawing the directions of the principal stress at all points, the envelopes to these may be traced ; these form a set of orthogonal curves throughout the stressed region, and are termed " Lines of principal stress," or Principal stress trajectories " (Fig. 3).

From the principal stress differences and the directions of principal stresses, the shear stress $\widehat{xy}$ in the plane xy can be calculated (Table 2). The longitudinal and transverse stresses $\widehat{xx}$, $\widehat{yy}$ can then be obtained by graphical integration (appendix 1 and Figs. 5 and 6). Finally from the longitudinal and transverse stresses the principal stresses are computed (Figs. 7, 8 and 9).

Apparatus, Models, Method of Loading

The Isoclinic and Isochromatic patterns were obtained on a photoelastic bench, with arrangements for plane and circular polariscope, and the Sénarmont

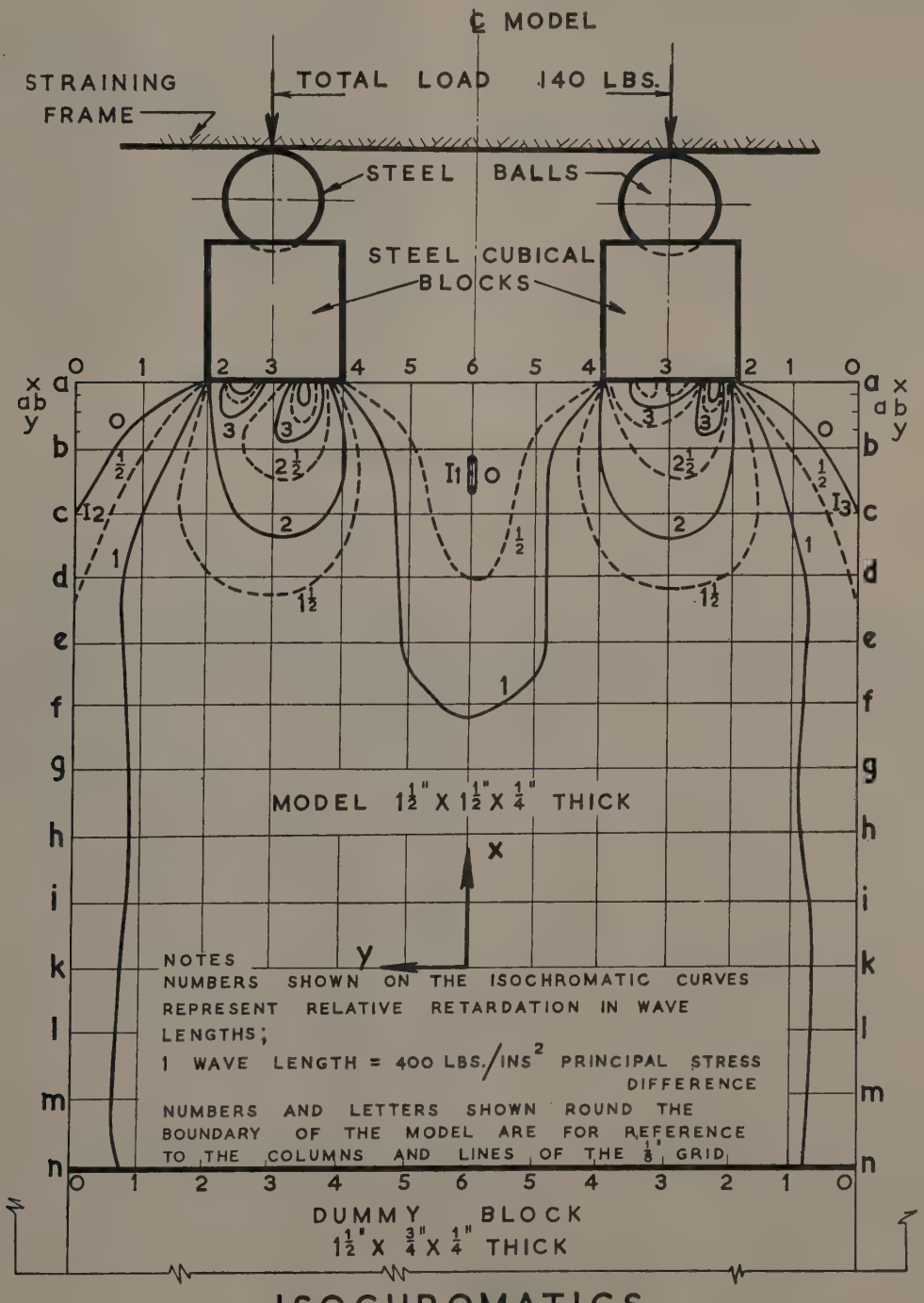

℄ MODEL
STRAINING FRAME
TOTAL LOAD 140 LBS.
STEEL BALLS
STEEL CUBICAL BLOCKS
MODEL 1½" X 1½" X ¼" THICK
NOTES
NUMBERS SHOWN ON THE ISOCHROMATIC CURVES REPRESENT RELATIVE RETARDATION IN WAVE LENGTHS;
1 WAVE LENGTH = 400 LBS./INS² PRINCIPAL STRESS DIFFERENCE
NUMBERS AND LETTERS SHOWN ROUND THE BOUNDARY OF THE MODEL ARE FOR REFERENCE TO THE COLUMNS AND LINES OF THE ⅛" GRID
DUMMY BLOCK
1½" X ¾" X ¼" THICK
ISOCHROMATICS
ONE WAVE LENGTH STRESS DIFFERENCE
HALF DO DO

Fig. 2

method was employed throughout all observations to establish fractional stress difference values, at $\frac{1}{8}$ in. grid and intermediate points.

White and mercury light were used, and where monochromatic light was needed, a green filter was inserted in the path of the beam of light to eliminate all colours except the green of the mercury arc.

Models used were of Columbia resin " CR/39," and " Perspex "; their finished dimensions were $1\frac{1}{2}$ in. × $1\frac{1}{2}$ in. by $\frac{1}{4}$ in. average thickness.

The Perspex models were intended to give a clearer isoclinic pattern, since perspex is of low photoelastic sensitivity and exhibits comparatively little fringe effect.

The load was applied on two steel balls which were housed in spherical recesses provided on steel blocks bearing on the loading face of the model, at $\frac{1}{4}$-depth points. These steel blocks were $\frac{1}{4}$ in.-cubes, and were packed with paper on the loading faces to apply a uniformly distributed load on the model.

A " dummy " block $1\frac{1}{2}$ in. × $\frac{3}{4}$ in. × $\frac{1}{4}$ in. thick supported the model to give uniform distribution of pressure and cork packing was placed between the arm of the straining frame and the " dummy " block for similar reasons. (Fig. 1).

The Isochromatics, Isoclinics, and Lines of Principal Stress

The isochromatics and isoclinics are shown o figures 2 and 4, respectively. They both exhib three " isotropic " points I_1, I_2 and I_3, i.e. points a which the principal stresses, and therefore all stresse are equal. Point I_1 lies on the centre line of the mode at just over $\frac{1}{8}$ in. distance from the loading edge points I_2 and I_3 are at identical positions, one on eac longitudinal boundary, at approximately $\frac{1}{4}$ in. distanc from the corners. At these latter points since on principal stress is normal to the boundary and vanishes all stresses will vanish.

On figure 3, the lines of principal stress are shown from which the sign of the boundary stresses can b determined. Thus, lines shown full have been take as the Q-lines of principal stress, and those show dotted as the P-lines of principal stress.

If then the convention is used that compressiv stresses are negative and tensile stresses are positive the stress difference $(Q—P)$ will be negative at a poin far enough from the loaded edge of the model, fo uniformity of stress distribution to exist, wher $P = O$ and Q is compressive. For reasons of con tinuity $(Q—P)$ does not change sign and is therefor negative everywhere in the model.

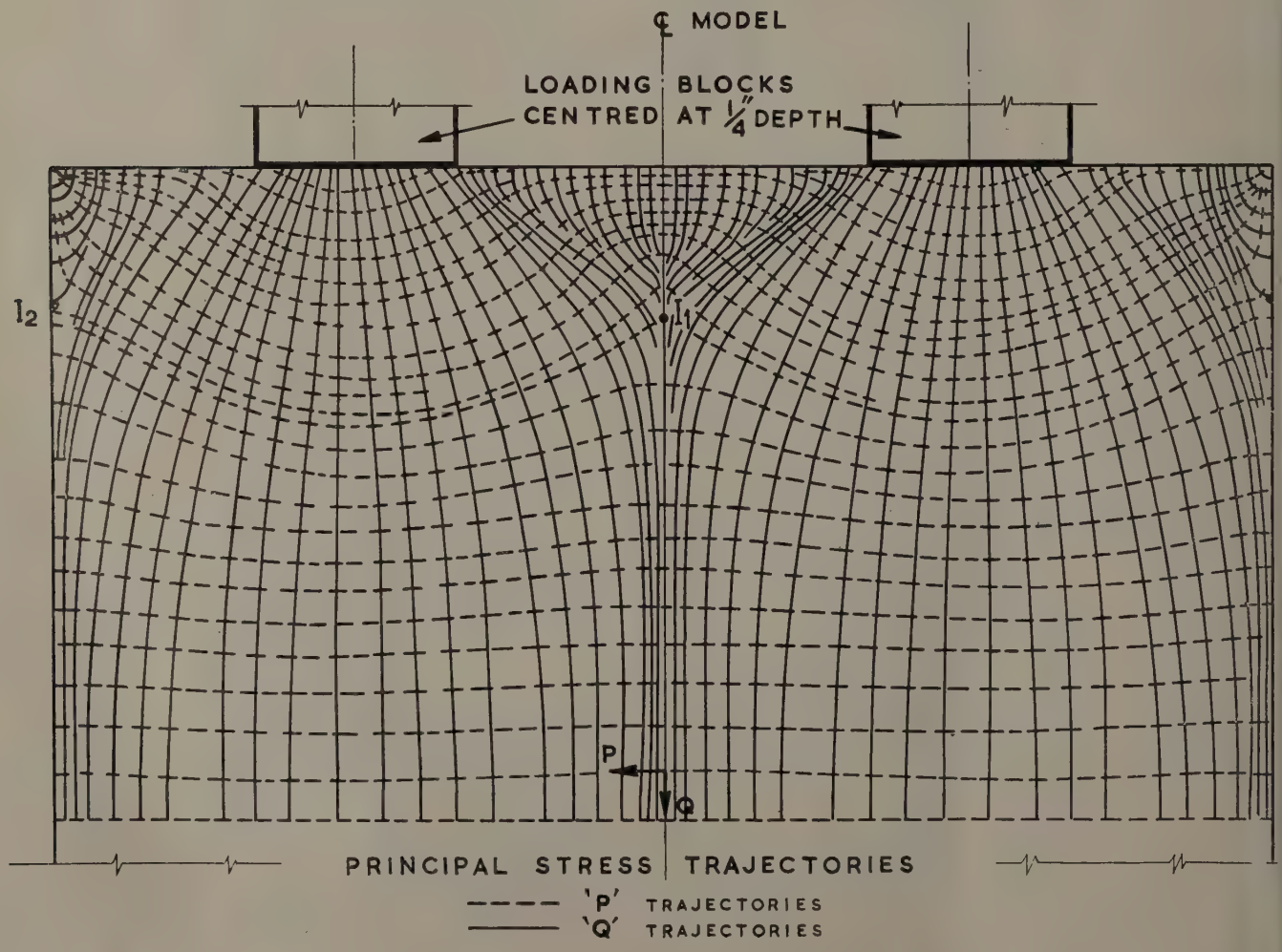


Fig. 3

Also, since the parts of the longitudinal faces above I_2 and I_3 are intersected at right angles by the Q-lines, hey are themselves P-lines. It follows that, as $= O$, since it is a normal stress to a free (non-loaded) oundary, P is positive. With similar reasoning the eneral result may be obtained that the boundary tresses are tensile on the loaded face everywhere xcept under the loads, and also tensile on the longiudinal faces for some distance from the loaded face, hanging to compressive stresses at the isotropic oints I_2 and I_3. These results were confirmed by Babinet compensator (4).

Both isoclinic and isochromatic patterns display he fact that about $\frac{3}{4}$ in. away from the loaded face of he model, the distribution of stress becomes uniform; his means that beyond a distance of half the depth f the beam there exists only uniform compression. his affords a method of establishing the "fringe alue" of the material (see below).

Relative Retardation and Fringe Value

The fringe value of a photoelastic material is in b./in.^{2} per inch thickness per fringe. This expresses he relation of simple proportion which has been xperimentally found to exist between the relative etardation introduced by the stressed material when he light wave, split into two components, has travelled hrough it, and the difference of the principal stresses t the point of travel.

The relative retardation observed at the points where only uniform compression exists, i.e. beyond half the depth, in the "C.R. 39" model is 167°, or $\frac{167}{180}$ in wavelengths.

If, therefore,

f = fringe value of the material,
t = thickness of the model = $\frac{1}{4}$ in., and
width of model = $1\frac{1}{2}$ in.,
total load applied = 140 lb.

Therefore

$$\frac{167}{180} \times \frac{f}{t} = \frac{140}{\frac{3}{2} \times t} \text{ and}$$

$f \triangleq 100$ lb./in^2 per inch per fringe

Then

The fringe value of the model $= \frac{f}{t} = 400$ lb./in.^{2} per fringe.

This means that, referring to figure 2, points on the fringe marked 1 have caused a relative retardation of one wavelength. The principal stress difference at these points is therefore 1×400 lb./in.^{2}; similarly for points on the fringes marked $1\frac{1}{2}$, 2, $2\frac{1}{2}$, etc.

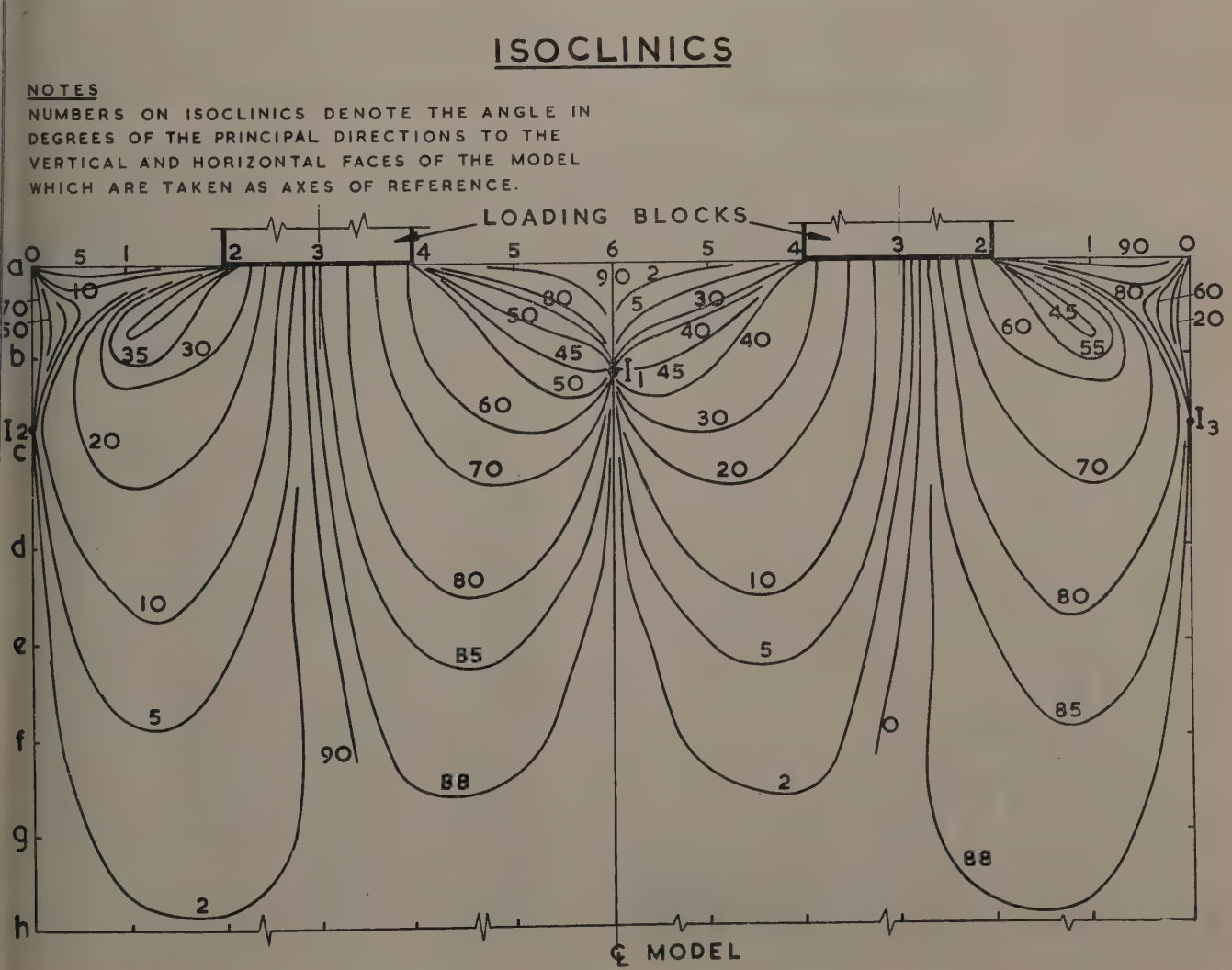

ISOCLINICS
NOTES
NUMBERS ON ISOCLINICS DENOTE THE ANGLE IN DEGREES OF THE PRINCIPAL DIRECTIONS TO THE VERTICAL AND HORIZONTAL FACES OF THE MODEL WHICH ARE TAKEN AS AXES OF REFERENCE.
LOADING BLOCKS
I1
I2
I3
℄ MODEL

Fig. 4

Observations

By using a plane polariscope arrangement, with analyser and polariser coupled, the isoclinic pattern was traced (Fig. 4).

The isochromatics for whole and half wavelengths (Fig. 2) were obtained by viewing the loaded model in a circular polariscope set up.

The Sénarmont method was employed throughout to obtain fractional values of the relative retardation at points of the $\frac{1}{8}$-in. grid (Table 1).

Conclusions

(*a*) The maximum shear stress and the absolute maximum principal stress occur immediately under, and on the centre line of, the loading steel cubes, i.e. the anchorages.

For a total post-tensioning load of 140 lb. equally applied by two cables, the maximum shear stress is approximately 750 lb./in.^{2}. The absolute maximum principal stress is approximately 1,500 lb./in.^{2} and is compressive.

The maximum principal tensile stress occurs near the loaded end face, between the anchorages and the centre line of the beam, and is approximately 230 lb./in.^{2}

These values expressed in terms of the uniform compression applied by the load, which is 374 lb/in.^{2}, are :—

Maximum shear stress
=2x uniform compression

Maximum principal compressive stress
=4x uniform compression

Maximum principal tensile stress
=0.6x uniform compression

The position and magnitude of the principal tensi stresses obtained here may be used to explain th anchorage failures, mentioned in the introductio which were observed on concrete beams with sol rectangular end blocks and cables anchored syn metrically about the centre line of the end section. I these beams the mild steel reinforcement provided the end anchorage would be considered inadequate compared with the values of tensile stresses give above, although the amount of steel used was calculate (as far as the author could ascertain) by the standar available methods for designing end blocks.

(*b*) The quantitative value of the distributio established may be seen by comparing values of th invariant function $\Theta = \widehat{xx} + \widehat{yy} = P + Q$, obtaine by two different methods, i.e. from $\widehat{xx}$ and $\widehat{yy}$ calculate by the shear slope[5] method, and from the values P and Q calculated from $\widehat{xx}$ and the observed values the principal stress difference.

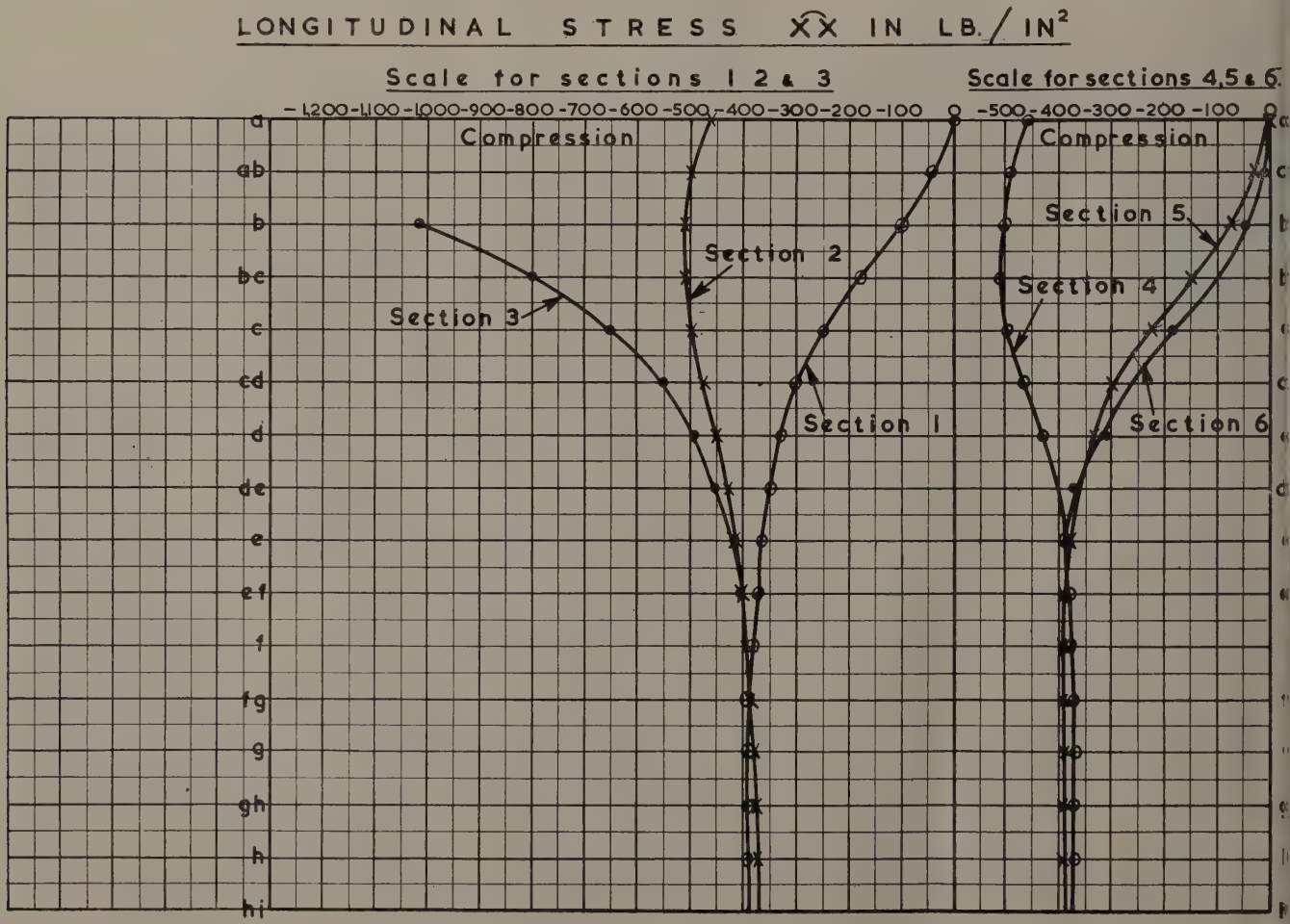


Fig. 5

Another check was made using the Lamé-Maxwell uations to calculate P and Q for the sections 3 and 6. lso, on integrating by the shear slope method along te longitudinal sections of the beam the value of the uiform compression arrived at agreed with the teoretical one within 6%. This was considered tisfactory.

(*c*) It must however be remembered that the actual istribution of stress in the end blocks of post-tensioned ams is a three-dimensional one, i.e. the stresses ry along the width of the beam as well as the depth d length. Experimental evidence now available om the author's three-dimensional investigation of e problem indicate the importance of this point. lso, the cable ducts cast in the concrete will affect the stribution of stress.

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Max. shear stress in lb./in.²

Transverse section ←	0	1	2	3	4	5	6	← Longitudinal section
a	0	33	270	550	210	115	110	
x	15	14	306	600	263	72	48	
ab	22	0	334	600	309	74	34	
y	23	50	367	637	365	100	19	
b	22	87	360	633	388	148	9	
bc	9	139	367	503	365	158	29	
c	10	178	334	427	334	161	61	
d	73	194	269	302	259	167	135	
e	120	200	239	252	239	192	180	
f	156	187	204	220	219	194	191	
g	171	187	187	187	187	186	186	
h	187	187	187	187	187	187	187	

Table 1

Notes :—

(*a*) The above values of max. shear stress were obtained from the observed values of the rel. retardation in wave lengths, see Introduction.

(*b*) It will be observed that a uniform state of stress occurs along the line *h*. This affords a method of calculating the fringe value of the material, (see text).

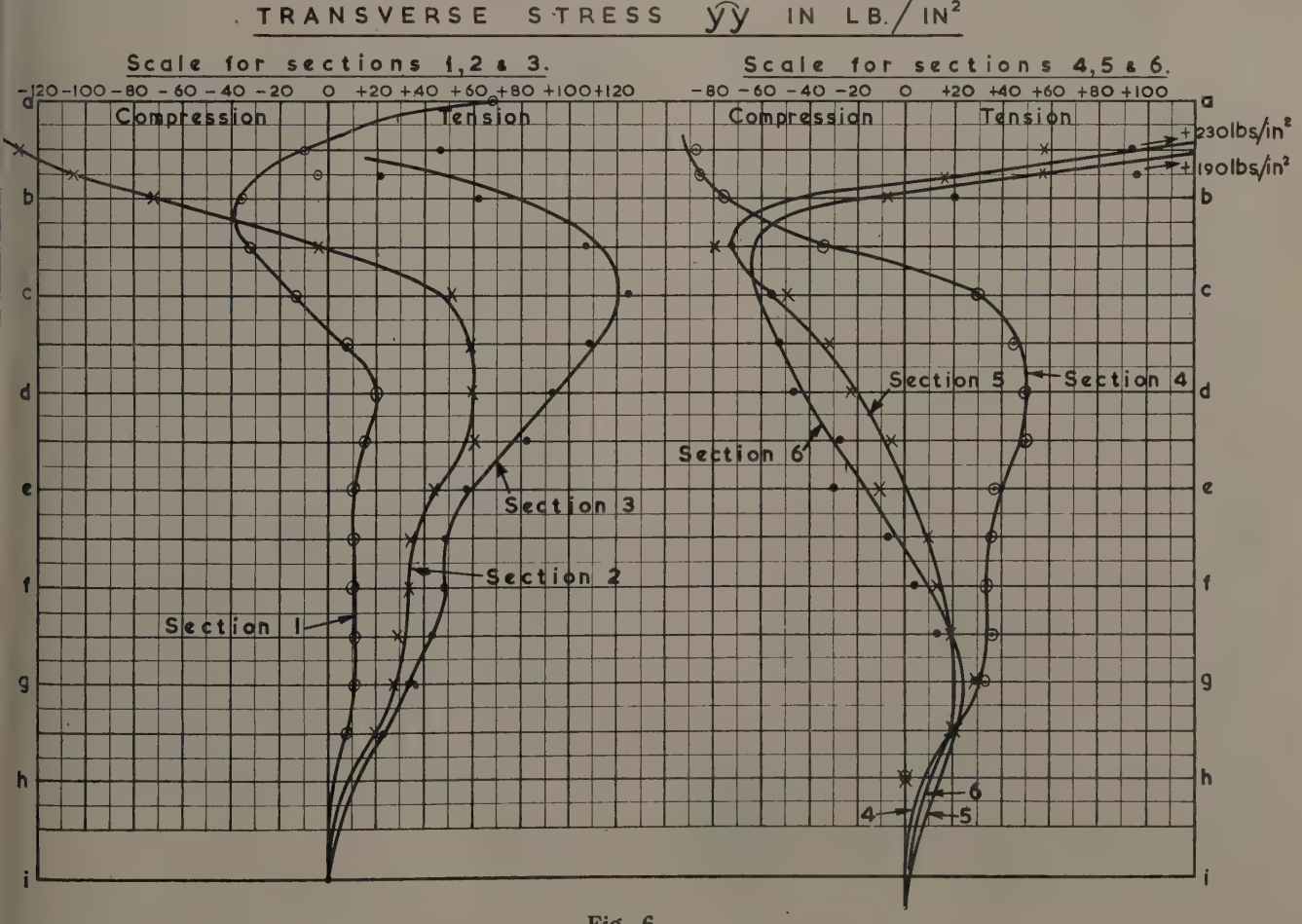


Fig. 6

Calculated Values of the Principal Stresses P and Q and the Invariant $\Theta = P + Q$

↓—Section	Values of P in lb./in.²							Values of Q in lb./in.²						
	Point 0	1	2	3	4	5	6	0	1	2	3	4	5	6
a	0	+120	—	—	+125	+230	+190	0	0	—	—	—740	0	0
b	0	+6	+93	+89	+122	+85	—30	+43	—167	—627	—1,176	—654	—210	—35
c	0	+51	+120	+145	+109	+29	—59	—20	—305	—547	—710	—558	—293	—177
d	0	+44	+77	+95	+68	—10	—13	—145	—344	—461	—509	—449	—343	—283
e	0	+33	+61	+67	+89	+8	+23	—240	—367	—417	—437	—389	—376	—337
f	0	—6	+17	+48	+63	+4	+11	—312	—379	—391	—392	—375	—384	—363
g	0	—11	—3	0	—1	—14	0	—342	—384	—376	—374	—374	—387	—372
h	0	0	0	0	0	0	0	—372						

Calculated values of $\widehat{xy}$

←—Transverse section	(+*ve*) shear stress in lb./in.²							(—*ve*) shear stress in lb./in.²						Longitudinal section—→
	0	0–1	1	1–2	2	2–3	3	3–4	4	4–5	5	5–6	6	
a	0	0	0	0	272	–	0	–	200	0	0	0	0	
x	0	13	20	80	288	427	0	493	248	107	53	30	0	
ab	0	14	42	116	292	340	0	395	289	143	69	30	0	
y	0	10	62	158	289	296	0	373	328	184	99	43	0	
b	0	18	82	185	274	250	0	304	328	225	130	45	0	
bc	0	24	115	195	220	157	0	191	275	230	145	54	0	
c	0	49	118	158	170	109	0	123	203	189	130	57	0	
cd	0	43	100	130	128	70	0	82	144	154	109	52	0	
d	0	43	89	107	95	52	0	52	98	108	83	40	0	
de	0	33	72	80	71	33	0	38	61	78	61	29	0	
e	0	30	52	58	53	30	0	21	36	49	40	19	0	
ef	0	25	44	45	38	19	0	16	25	24	20	13	0	
f	0	19	30	34	28	15	0	7	18	20	10	3	0	
fg	0	16	20	20	17	3	0	0	2	3	3	0	0	
g	0	7	13	14	8	0	0	0	0	0	0	0	0	
gh	0	0	3	3	3	0	0	0	0	0	0	0	0	
h	0	0	0	0	0	0	0	0	0	0	0	0	0	

Table 2

For the derivation of values in this table see Appendix 1.

↓—Section	Values of $\Theta = P + Q$ in lb./in²						
	Point 0	1	2	3	4	5	6
a	0	—	—	—	—	—	+190
b	+43	—161	—534	—1,087	—532	—125	—65
c	—20	—254	—427	—565	—449	—264	—236
d	—145	—300	—384	—414	—381	—353	—296
e	—240	—334	—356	—370	—300	—368	—314
f	—312	—385	—374	—344	—312	—380	—352
g	—342	—395	—379	—374	—375	—401	—372
h	—372	—	—	—	—	—	—

Table 3

Notes :—

(*a*) *Sign Convention* :—
(+*ve*) values indicate tension.
(—*ve*) ,, ,, compression.

(*b*) Values of P, Q and $P + Q$ for all points except those of column '6,' were computed from values of $\widehat{xx}$.

Values of P, Q and $P + Q$ in column '6' were computed by direct integration employing the Lamé-Maxwell equations.

Calculation of the Invariant $\Theta = \widehat{xx} + \widehat{yy}$, from values obtained by the shear slope method

↓—Section	Section 1			2			3			4			5			6		
	$\widehat{xx}$	$\widehat{yy}$	Θ	$\widehat{xx}$	$\widehat{yy}$	Θ	$\widehat{xx}$	$\widehat{yy}$	Θ	$\widehat{xx}$	$\widehat{yy}$	Θ	$\widehat{xx}$	$\widehat{yy}$	Θ	$\widehat{xx}$	$\widehat{yy}$	Θ
a																		
b	—103	—36	—139	—507	—72	—579	—1,012	+62	—950	—505	—76	—581	—73	—40	—113	—42	—18	—60
c	—251	—13	—264	—492	+51	—441	—649	+124	—525	—493	+30	—463	—226	—49	—275	—185	—60	—245
d	—324	+20	—304	—445	+59	—386	—492	+93	—399	—426	+50	—376	—326	—23	—349	—305	—40	—345
e	—359	+10	—349	—411	+44	—367	—420	+58	—362	—385	+43	—342	—372	0	—372	—370	—16	—386
f	—377	+10	—367	—389	+34	—345	—387	+48	—339	—375	+36	—339	—384	+13	—371	—388	+10	—378
g	—384	+11	—373	—376	+28	—348	—376	+33	—343	—374	+33	—341	—387	+20	—367	—382	+24	—358
h	—383	+4	—379	—374	+10	—364	—374	+8	—366	—374	+9	—365	—387	+12	—376	—382	+9	—373

Note :—Values of Θ obtained in this table may be compared with those in table 6, as a check of the quantitative value of this work.

Table 4

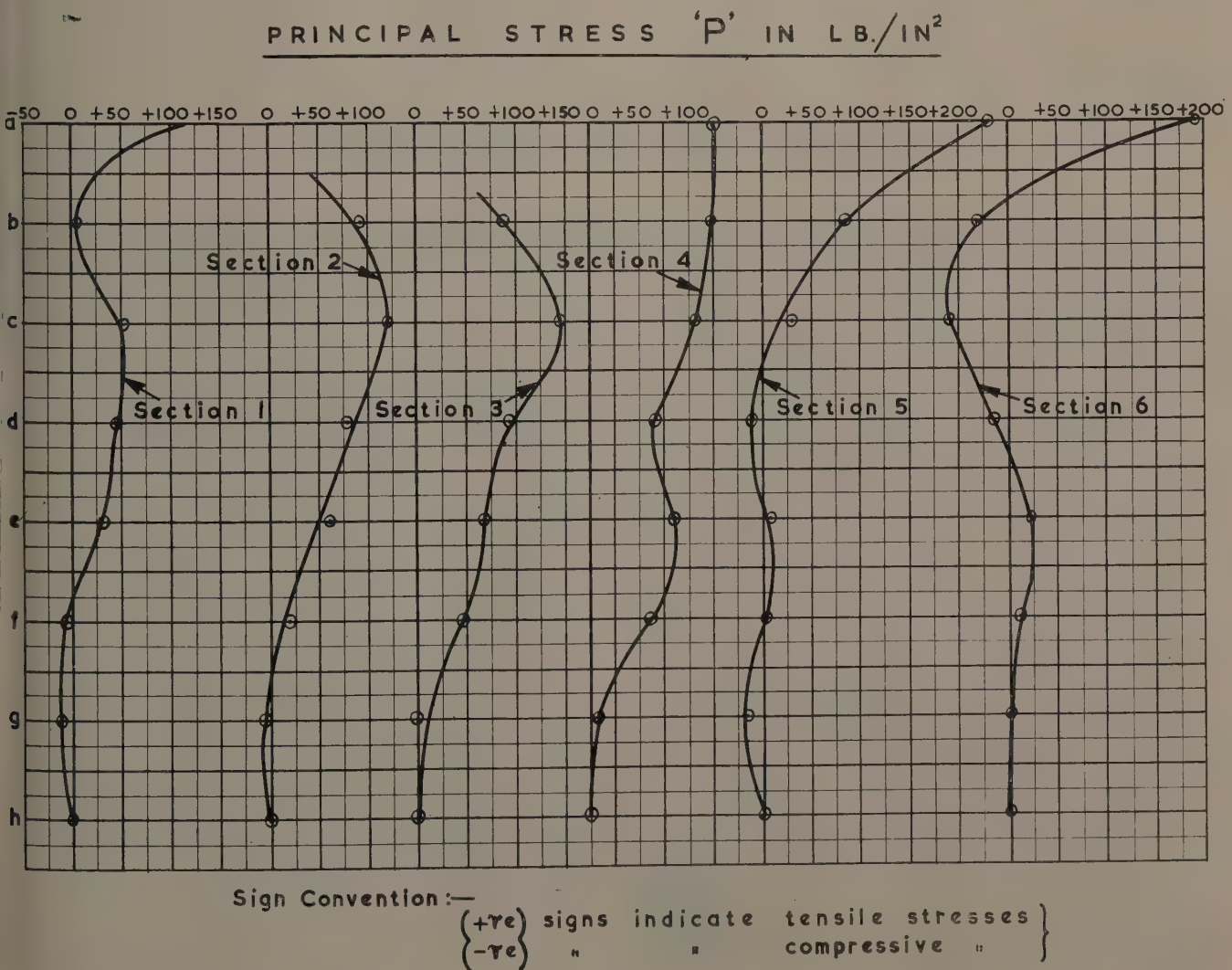


Fig. 7

Integration along longitudinal section ' 3,' assumed as a principal stress trajectory, by the Lamé-Maxwell equations

	1	2	3	4	5	6	7	8	9	10
	$Q-P$ lb./ in.2	$\triangle\theta$ rad.	$\triangle s$ ins.	$\triangle s1$ ins.	$\frac{\triangle s1}{\triangle s} \times \triangle\theta$	$\triangle Q$ lb./ in.2	$\triangle Q$ mean	Q lb./in.	P lb./ in.2	$P+Q$ lb./in.2
a										
b	—1,265	.175	.285	.8	.492	622		—1,176	+89	—1,087
							466			
c	—855	.175	.38	.8	.363	310		—710	+145	—565
							201			
d	—604	.07	.365	.8	.153	92		—509	+95	—414
							72			
e	—504	.07	.53	.8	.106	53		—437	+67	—370
							45			
f	—440	.07	.685	.8	.082	36		—392	+48	—344
							18			
g	—375			0	0	0		—374	0	—374
h	—374								0	

Table 5

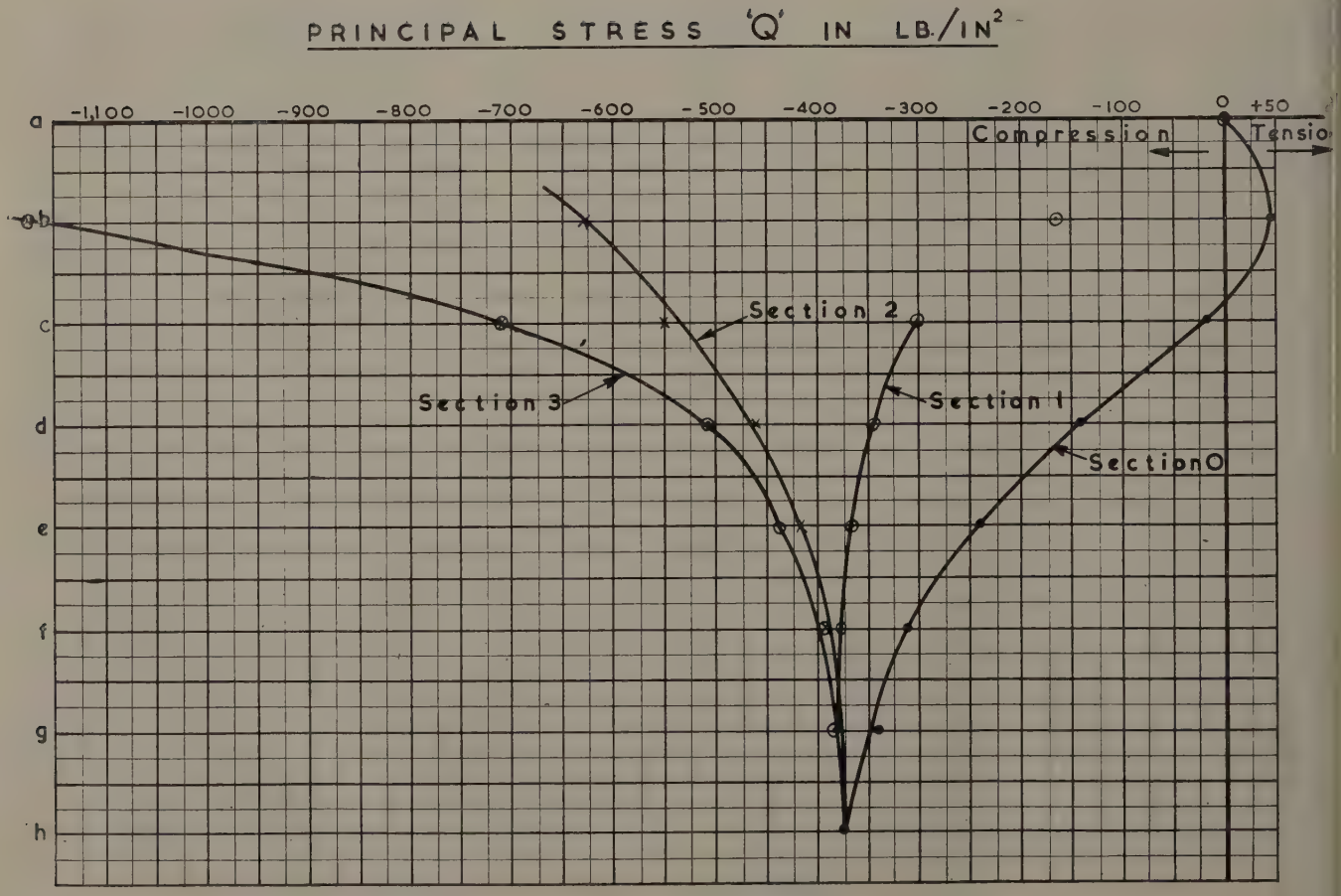


Fig. 8

Shear slope Integration on Longitudinal Section 1—2

	S1	*S2*	$\triangle S$	*S.M.*	*P*	*A1*	*A2*	*A*	$\widehat{xx}$ lb./in.^{2}
a	0	272	272	1	272				0
ab	42	292	250	4	1,000				
b	82	274	192	1	192	1,464	1,464	244	—244
bc	115	220	105	4	420				
c	118	170	52	1	52	664	2,128	355	—355
cd	105	128	23	4	92				
d	89	95	6	1	6	150	2,278	380	—380
de	72	71	—1	4	—4				
e	55	53	—2	1	—2	0	2,278	380	—380
ef	44	38	—6	4	—24				
f	30	28	—2	1	—2	—28	2,250	375	—375
fg	20	17	—3	4	—12				
g	13	10	—3	1	—3	—17	2,233	372	—372
gh	3	3	0	4	0				
h	0	0	0	1	0	—3	2,230	372	—372

Notes :—

1. *S1*, *S2* are the values of shear stress $\widehat{xy}$.

2. Simson's rule has been used for computing the areas, i.e.

$$A = \tfrac{1}{3} \times \frac{\triangle x}{\triangle y} \times \left\{ \begin{matrix} 1 \times \triangle S1 + 4 \times \triangle S2 + 2 \triangle S3 \\ + \ldots + 4 \times \triangle S_{n-1} + 1 \times \triangle Sn \end{matrix} \right\}$$

where n = number of ordinates,

$\triangle x = \frac{1}{16}$ in. $\triangle y = \frac{1}{8}$ in.

Therefore if the bracket is denoted by *A2*,

$$A = \tfrac{1}{3} \times \frac{\frac{1}{16}}{\frac{1}{8}} \times A2 = \frac{A2}{6}$$

3. *Explanation of Symbols* :

S.M. = Simson's Multiplier.
S = Shear stress difference.
P = (*S.M.*) × *S*.
A1 = Non-Reduced Area.
A2 = Reduced Area to the point concerned.

Table 6

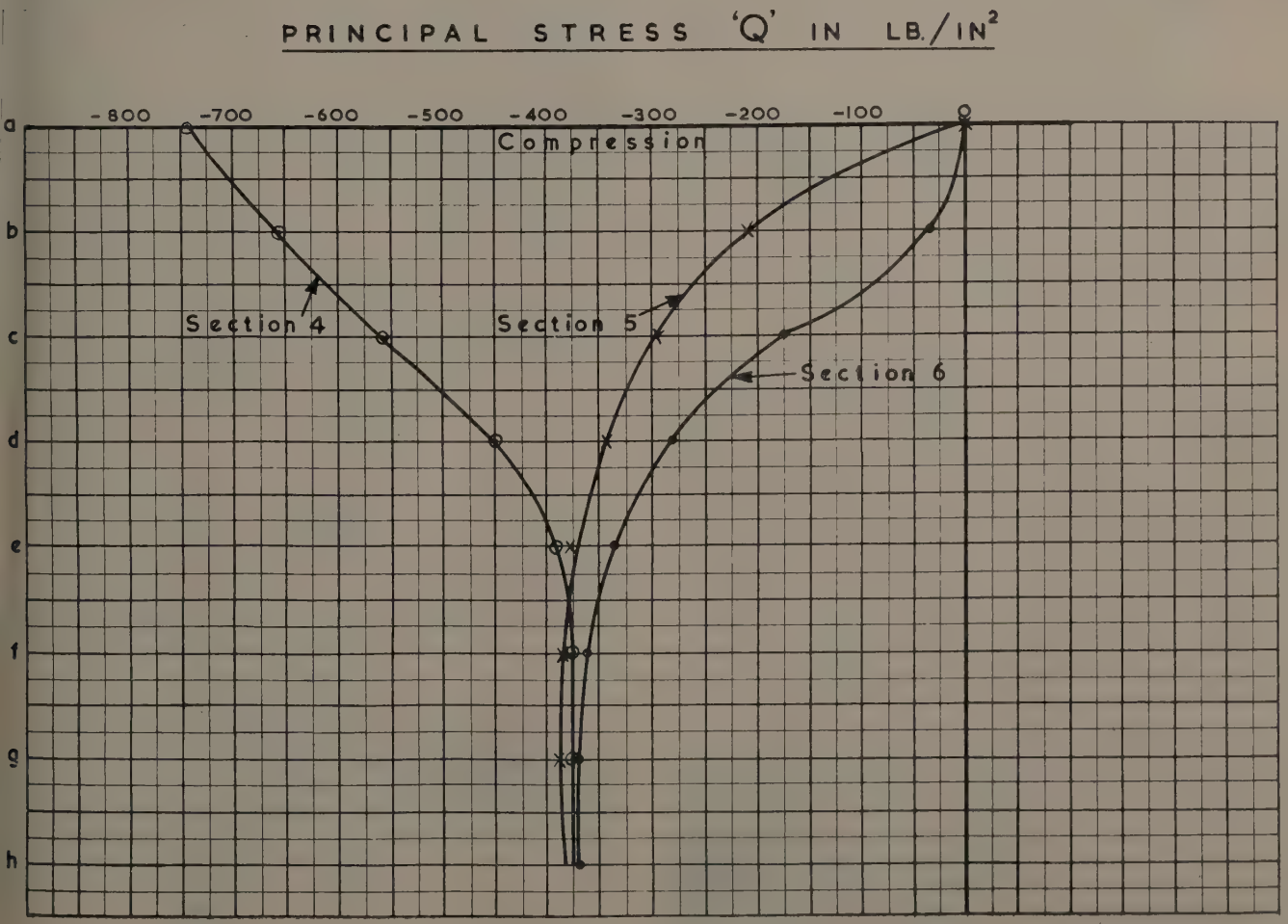


Fig 9.

APPENDIX I

Summary of Notation

$\widehat{xx}$	Transverse stress in the direction x
$\widehat{yy}$	Longitudinal stress in the direction y
$\widehat{xy}$	Shear stress
$\widehat{nn}, \widehat{ss}, \widehat{ns}$	Stresses corresponding to $\widehat{xx}$, $\widehat{yy}$, $\widehat{xy}$ with respect to axes O (n, s)
P, Q	Principal stresses
f	Fringe value
t	Thickness of model
ρ_4, s	Radius of curvature and arc respectively, of the P-principal stress trajectory.
ρ_1, s_1	Radius of curvature and arc, respectively, of the Q-principal stress trajectory.
θ	Isoclinic angle
r, θ	Polar co-ordinates
r	Relative retardation, or, principal stress difference
p	Uniformly distributed pressure
Θ	Stress combination of the form $\widehat{xx}+\widehat{yy}$
Φ	Stress combination of the form $\widehat{xx}-\widehat{yy}+2i\widehat{xy}$

APPENDIX II

Calculation of Principal Stresses from Observation

(*a*) *Integration by the Lamé-Maxwell equations along a line of principal stress.*[4]

The Lamé-Maxwell equations are :—

$$\left.\begin{aligned}\frac{\partial P}{\partial s}+\frac{P-Q}{\rho_1}&=0\\ \frac{\partial Q}{\partial s_1}+\frac{P-Q}{\rho}&=0\end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where P, Q are the values of principal stresses at the point considered ; s, s_1 are the arcs of the P and Q-principal stress trajectories respectively and ρ_1 ρ_1 the radii of curvature of the P and Q-principal stress trajectories respectively.

The second of the above equations can be written, if second order small quantities are neglected :—

$$\Delta Q=(Q-P)\times\frac{\Delta\theta}{\Delta s}\times\Delta s_1$$

where θ is the angle of rotation of the tangent to the P-trajectory.

This equation may be used to obtain the values of Q at points on a Q-trajectory, if the integration is started from a point where Q is known, e.g. from a boundary normal to the direction of Q, where $Q=o$. Having obtained Q, the other stress may be calculated directly from the observed value of $P-Q$, at the point, (Table 1).

On table 5 an example of the computation involved in " splitting " the stresses by this method is given for the longitudinal section 3 of the $\frac{1}{8}$ in. grid, (Fig. 2), which was assumed as nearly a principal stress trajectory.

It will be seen that integration was commenced in this case from the known value of the principal stress Q at the line g on the grid where uniform compression exists and the other principal stress $P=o$ and :—

$$Q-P=Q=\frac{140}{\frac{3}{2}\times\frac{1}{4}}=374\text{ lb./in.}^2\text{ (Column 8, Table 5).}$$

(*b*) *Integration by the " shear-slope " method.*[5]

(*I*) *The sign of shear ; shear force in any direction*

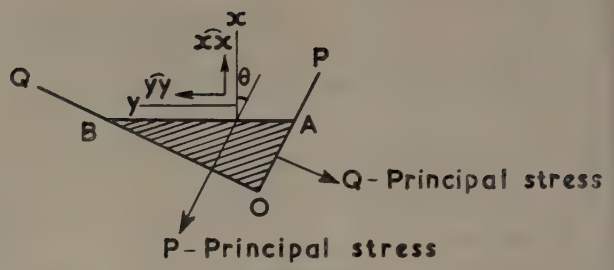


Fig. 10

Consider the triangular prism, OAB, of unit thickness, where OP, OQ are the principal directions, and let it be in equilibrium under the action of forces shown, Fig. 10. Let $\widehat{xx}$, $\widehat{yy}$, $\widehat{xy}$ be the mean stresses along the thickness of the model.

Equating forces in direction x,

$$\left.\begin{aligned}\widehat{xx}(AB)&=P(OB)\cos\theta+Q(OA)\sin\theta\ ;\\ \therefore\ \widehat{xx}&=P\cos^2\theta+Q\sin^2\theta\\ \text{Similarly,}\quad \widehat{yy}&=P\sin^2\theta+Q\cos^2\theta\end{aligned}\right\} \quad . \quad (2)$$

Also, equating forces in direction y,

$$\widehat{xy}=\frac{Q-P}{2}\sin^2\theta \quad . \quad . \quad (3)$$

This expression was used to calculate the values of $\widehat{xy}$ tabulated in table 2.

The normal stresses have been shown as tensions above which are conventionally taken positive, while compressive stresses would be negative and would introduce the corresponding change of sign in formulae (2) and (3).

Hence, for all points to the left of longitudinal section 3 :

$$\widehat{xy}=\frac{P-Q}{2}\sin 2\theta>O,$$

if compressive stresses are taken negative and $Q-P<O$; Fig. 11 (compare with Fig. 3)

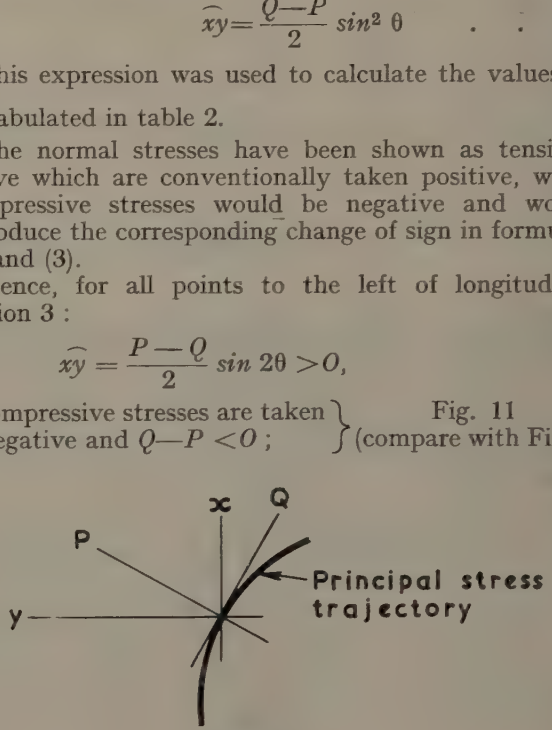


Fig. 11

For all points to the right of longitudinal section " 3 "

$$\widehat{xy}=\frac{Q-P}{2}\sin 2\theta<O\ ;$$ Fig. 12 (compare with Fig. 3)

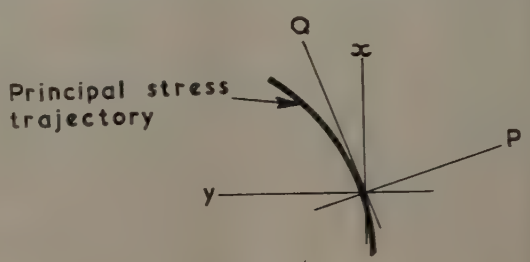


Fig. 12

(II) The equations of equilibrium in Cartesian co-ordinates give :—

$$\left.\begin{aligned}\frac{\partial \widehat{xx}}{\partial x}+\frac{\partial \widehat{xy}}{\partial y}&=O\\ \frac{\partial \widehat{yy}}{\partial y}+\frac{\partial \widehat{xy}}{\partial x}&=O\end{aligned}\right\},$$

where $\widehat{xx}$, $\widehat{yy}$, $\widehat{xy}$ denote as before the mean stresses along the thickness of the model.

The first of these equations may be written as :—

$$\partial \widehat{xx}=\frac{1}{\partial y}(\partial \widehat{xy})\,\partial x$$

Integrating from A to B, Fig. 13

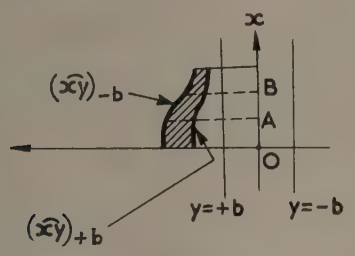


Fig. 13

Formula (4) has been used to obtain the values of $\widehat{xx}$, from which graphs on Fig. 5 have been plotted ; the integration for the sections 1, 5 and 6 was started from the loaded face i.e. section " a " since these sections have free points on boundary " a," and therefore $\widehat{xx}=o$ there.

For sections 2, 3 and 4, integration was started from the bottom, i.e. section " h " where $\widehat{xx}$ = uniform compression.

On table 6 an example of calculating $\widehat{xx}$ by this method is given for the longitudinal section halfway between section 1 and 2. Integration here was started from the free boundary on the loaded face of the model where $\widehat{xx}=o$, since it is normal to the free boundary.

A similar formula to (4) was used to obtain the values of $\widehat{yy}$, from which the graphs on figure 6 were plotted.

(III) From (2) : $P\cos^2\theta+Q\sin^2\theta=\widehat{xx}$;
Also from photoelastic observations :

$$P-Q=r,\ say\ ;$$

Solving these simultaneous equations for P and Q,

$$\left.\begin{aligned}P&=\widehat{xx}+r\sin^2\theta\\ Q&=P-r\end{aligned}\right\}\quad\ldots\quad(5)$$

Expressions (5) were used to obtain the values of P and Q shown in Table 3.

APPENDIX III

Analytical Solution for Stress Distribution

Constant pressure strips on a half-plane boundary.

The solution for a uniformly distributed pressure strip applied on a half-plane boundary, can be built up by superposing a number of solutions of a very simple type which are derived from the Airy stress function.

$$X=-\frac{P}{2\pi}r^2\theta=-\frac{P}{2\pi}(x^2+y^2)\tan^{-1}\left(\frac{y}{x}\right)$$

The half-plane is the upper half-plane $y\geq o$ (Fig. 14) and it is readily found that the stresses are :

$$\left.\begin{aligned}\widehat{xx}&=\frac{\partial^2X}{\partial y^2}=-\frac{p}{2\pi}(\sin 2\theta+2\theta)\\ \widehat{yy}&=\frac{\partial^2X}{\partial x^2}=\frac{p}{2\pi}(\sin 2\theta-2\theta)\\ \widehat{xy}&=-\frac{\partial^2X}{\partial x\partial y}=\frac{p}{2\pi}\cos 2\theta\end{aligned}\right\}\quad.\quad(6)$$

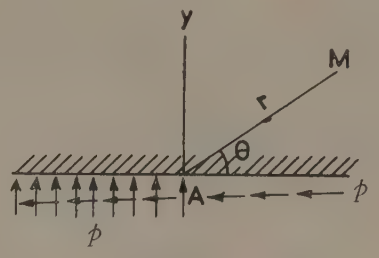


Fig. 14

and on the boundary $y=o$ ($\theta=o$ and $\theta=\pi$)

$\widehat{yy}=o$ for $\theta=o$, and is $-p$ for $\theta=\pi$.

$\widehat{xy}=\frac{p}{2\pi}$ along $y=O$.

so this simple solution corresponds to a normal pressure p along the negative x-axis, zero normal traction along the positive x-axis, and a constant shear $\frac{p}{2\pi}$ along the whole x-axis.

Pressure on a single strip of the half-plane.

Suppose on this solution with origin A, a second solution with origin B with the sign of p changed. This gives normal pressure p over the strip AB simply : (Fig. 15).

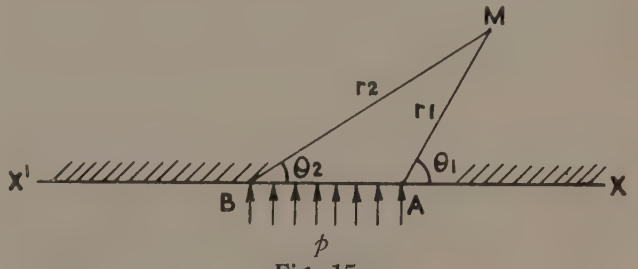


Fig. 15

and the stresses in the half-plane are, from (6)

$$\left.\begin{aligned}\widehat{xx}&=-\frac{p}{2\pi}(\sin 2\theta_1-\sin 2\theta_2+2\theta_1-2\theta_2)\\ \widehat{yy}&=\frac{p}{2\pi}(\sin 2\theta_1-\sin 2\theta_2-2\theta_1+2\theta_2)\\ \widehat{xy}&=\frac{p}{2\pi}(\cos 2\theta_1-\cos 2\theta_2)\end{aligned}\right\}\quad.\quad(7)$$

Equal Pressures on Two equal strips.

A particular case of interest is when there are two of these pressure strips, under the same pressure p, each of the same length $2c$, with their centres at a distance $6c$ apart as in Fig. 16.

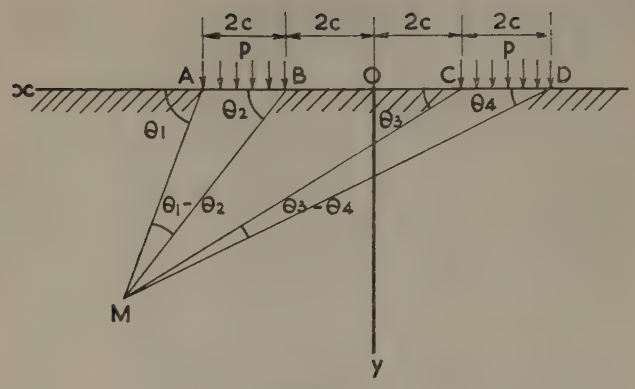


Fig. 16

The stresses in the half-plane are from (7)

$$\left.\begin{aligned}
\widehat{xx} &= -\frac{P}{2\pi}\left\{\begin{matrix} sin\,2\theta_1 - sin\,2\theta_2 + sin\,2\theta_3 - sin\,2\theta_4 \\ +2\theta_1 - 2\theta_2 + 2\theta_3 - 2\theta_4 \end{matrix}\right\} \\
\widehat{yy} &= \frac{P}{2\pi}\left\{\begin{matrix} sin\,2\theta_1 - sin\,2\theta_2 + sin\,2\theta_3 - sin\,2\theta_4 \\ -2\theta_1 + 2\theta_2 - 2\theta_3 + 2\theta_4 \end{matrix}\right\} \\
\widehat{xy} &= \frac{P}{2\pi}\left\{cos\,2\theta_1 - cos\,2\theta_2 + cos\,2\theta_3 - cos\,2\theta_4\right\}
\end{aligned}\right\} \quad (8)$$

where the angles are as shown in Fig. 16.

The Principal Stresses and the Photoelastic Curves.

A simple way of carrying out the transformation to principal directions follows from expressing the stresses in combinations Θ, Φ,

where

$$\Theta = \widehat{xx} + \widehat{yy}, \qquad \Phi = \widehat{xx} - \widehat{yy} + 2\,i\,\widehat{xy} \quad (9)$$

If at any point O the axes $O\,(x, y)$ are turned through an angle α, (Fig. 17), clockwise, to give axes $O\,(n, s)$, and the corresponding stress combinations are :—

$$\Theta^1 = \widehat{nn} + \widehat{ss}, \quad \Phi^1 = \widehat{nn} - \widehat{ss} + 2\,i\,\widehat{ns};$$

then the equations of transformation are :—

$$\Theta = \Theta^1, \qquad \Phi = \Phi^1\,e^{-2i\alpha} \quad . \quad (10)$$

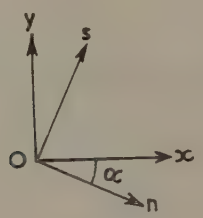


Fig. 17

If the directions n, s are those of the principal stresses P, Q, then :

$$\Theta^1 = P + Q, \qquad \Phi^1 = P - Q;$$

so,

$$P + Q = \widehat{xx} + \widehat{yy}, \quad P - Q = e^{2i\alpha}(\widehat{xx} - \widehat{yy} + 2i\widehat{xy})$$

The photoelastic curves of importance are the isochromatics $P-Q$ = constant, the isoclinics α = constant, and the isopachics $P+Q$ = constant ; or,

mod Φ = constant, *arg* Φ = constant, and Θ = constant, respectively.

Here, from (8) and (9) :

$$\left.\begin{aligned}
\Phi &= \frac{ip}{\pi}\left\{\begin{matrix} 2i\theta_1 & 2i\theta_2 & 2i\theta_3 & 2i\theta_4 \\ e & -e & +e & -e \end{matrix}\right\} \\
\Theta &= \frac{-2p}{\pi}\left\{\theta_1 - \theta_2 + \theta_3 - \theta_4\right\}
\end{aligned}\right\} \quad . \quad (11)$$

The isotropic points, i.e. points at which the principal stresses and therefore all stresses are equal are given by $P - Q = o$, or, $|\Phi| = o$.

The Isotropic Points on the line of Symmetry.

On the line of symmetry $x = o$, we have

$$\theta_1 + \theta_4 = \pi, \ \theta_2 + \theta_3 = \pi, \text{ leading to}$$

$$\begin{aligned}
\Phi &= \frac{ip}{\pi}\left\{\begin{matrix} 2i\theta_1 & -2i\theta_1 & 2i\theta_2 & -2i\theta_2 \\ e & -e & -e & +e \end{matrix}\right\} \\
&= \frac{-2p}{\pi}\,(sin\,2\theta_1 - sin\,2\theta_2)
\end{aligned}$$

$$\therefore \quad \Phi = \frac{-4p}{\pi}\,sin\,(\theta_1 - \theta_2)\,cos\,(\theta_1 + \theta_2) \quad . \quad (12)$$

Now along Oy, $\theta_1 - \theta_2$ vanishes at O and again at infinity, whereas $\theta_1 + \theta_2$ decreases from 2π at O to π at infinity, so the isotropics in the finite region are at the origin O and at the point $\theta_1 + \theta_2 = \dfrac{3\pi}{2}$;

hence $\tan\theta_1\,tan\,\theta_2 = 1$; or, $\left(\dfrac{-y}{4c}\right)\left(\dfrac{-y}{2c}\right) = 1$;

or,

$$y^2 = 8c^2, \quad \text{i.e.} \quad y = 2\sqrt{2}c.$$

Hence, in addition to the isotropic point at the origin there is an isotropic point on the line of symmetry at a distance $2\sqrt{2}c$ from the loaded boundary.

In the actual model $c = \frac{1}{8}$ in. and the isotropic point was observed at $y = 0.20$ ins., as compared with the theoretical value which gives $y = 0.35$ ins., the discrepancy being due to the presence of boundaries at a finite distance in the actual model. The same explanation may be suggested for the difference between the experimental and theoretical results obtained for the loaded boundary and the isotropic point at the origin.

Isoclinics.

From

$$\Phi = \left(P - Q\right)_e^{-2i\alpha}$$

$$arg\ \Phi = -2\alpha, \text{ or,}$$

$$tan\,2\alpha = \frac{cos\,2\theta_1 - cos\,2\theta_2 + cos\,2\theta_3 - cos\,2\theta_4}{sin\,2\theta_1 - sin\,2\theta_2 + sin\,2\theta_3 - sin\,2\theta_4} \quad . \quad (13)$$

In Cartesian coordinates, we can write this

$$tan\,2\alpha = \frac{2xy\left\{r^4 + 16c^2(x^2 - y^2) - 224c^4\right\}}{\begin{matrix}(12c^2 - x^2 + y^2)r^4 + 96c^4(x^2 - y^2) \\ +64c^2x^2y^2 - 512c^6\end{matrix}} \quad . \quad (14)$$

or putting $tan\,2a = t$, in polar coordinates the equation of the isoclinics is,

$$r^6(\sin 2\theta + t\cos 2\theta) + 4c^2r^4\left[2\sin 4\theta - t(5 - 2\cos 4\theta)\right] - 32c^4r^2(7\sin 2\theta + 3t\cos 2\theta) + 512tc^6 = o \quad . \quad (15)$$

On the line of symmetry Oy, $\theta = \frac{\pi}{2}$, so the points at which isoclinics cross the y-axis are given by :

$$r^6 + 12c^2r^4 - 96c^4r^2 - 512c^6 = o, \quad \text{for } t \neq O.$$

i.e. by : $(r^2 - 8c^2)(r^2 + 16c^2)(r^2 + 4c^2) = O \quad . \quad . \quad (16)$

leading to the point $r = y = 2\sqrt{2}c$, one of the two isotropic points found previously, all the isoclinics passing through this point. The isoclinic $t = o$ i.e. the O^0—isoclinic, is given by

$$xy(r^4 + 16c^2r^2\cos 2\theta - 224c^4) = o$$

i.e. $x = o$, $y = o$, and $r^4 + 16c^2r^2\cos 2\theta - 224c^4 = o \quad . \quad (17)$

Comparing with figure 4, $x = o$, $y = o$ are obtained experimentally as O°—isoclinics.

The remaining of equations (17) gives another O^0—isoclinic which passes very nearly through the centre lines of the pressure strips, as it may be seen by putting $\theta = O$, or, π when $r \triangleq \pm 3c$,

$$\text{For } \theta = \frac{\pi}{2}, \; r \triangleq 5c,$$

and this gives the point at which this O^0—isoclinic crosses the line of symmetry ; this compares with the corresponding O°—isoclinic (Fig. 4) passing through the centre lines of the loading blocks and crossing the centre lines of the model at $r \triangleq 1$ in. In the actual model $5c = \frac{5}{8}$ in., and the discrepancy is again due to the existence of the longitudinal boundaries of the model.

Book Reviews

Steel Designers' Manual, by Charles S. Gray, Lewis E. Kent, W. A. Mitchell and C. Bernard Godfrey. (Crosby Lockwood; London, 1955.). $9\frac{3}{4}$ in. × 6 in., pp. 909 + vii ; 50s. net.

This is an important and authoritative book. Written by four authors, each a corporate member of the Institution, it has been " Prepared for the British Steel Producers' Conference in conjunction with the British Iron and Steel Federation ; and these two corporate sponsors are not just names on the title page ; their interests were represented by a Committee " who drew up an outline of the Manual which has been of considerable assistance," as the authors acknowledge in their introduction. This active support of the Conference and the Federation sets a seal of authority on the work of the authors without detracting from their joint and several responsibility for the contents of the Manual, in respect both of selection of material and method of detailed presentation.

There are 49 chapters in these nine hundred pages, and each covers a particular subject of interest to the steel designer. Some of the chapters run to little more than a page or two, like the first which is a brief re-statement of the " ordinary beam theory," in a page and a half, whilst the longest runs to over a hundred on " modern methods of structural analysis," covering " moment distribution," " column analogy," " slope deflection," and " semi-graphical integration." A chapter on flat plates was contributed by C. C. Pounder, the one on " beams in torsion " by W. Fisher Cassie and one on " plastic theory " is abridged from a B.C.S.A. publication written by F. A. Partridge.

The emphasis is on rigid frames, for although trusses are noticed to the extent of an example of the Williot-Mohr construction for deflections and an introduction to influence lines, the design of bridge girders seems to be beyond the scope of the Manual. Rigid frame theory and practice are very well developed, and designers everywhere will be glad to have this ready reference to over a hundred pages of formulae and design charts, the majority of which have been extracted from Kleinlogel's many volumes of ready-to-use formulas presented here in much more intelligible guise than in the German texts.

Although the Manual is " Not a text-book of structural theory," the design methods are presented with adequate explanation, but no space is wasted on academic argument or justification, since the dominant theme is not why or wherefore but how to build in steel. The keyword is information rather than derivation or discussion, and in addition to the frame formulae there are many pages of useful tables, with numerous relevant extracts from British Standards and Building By-laws. The production of the book reaches a very high standard, and the printers, William Clowes and Sons Ltd., deserve special mention. For a volume of this weight and quality the price is remarkably low. Since it also goes a very long way towards complete achievement of the sponsors' and the authors' objective—bridging the gap between theory of design and its practical application—it can be warmly recommended to all structural engineers with an interest in steelwork.

E. H. B.

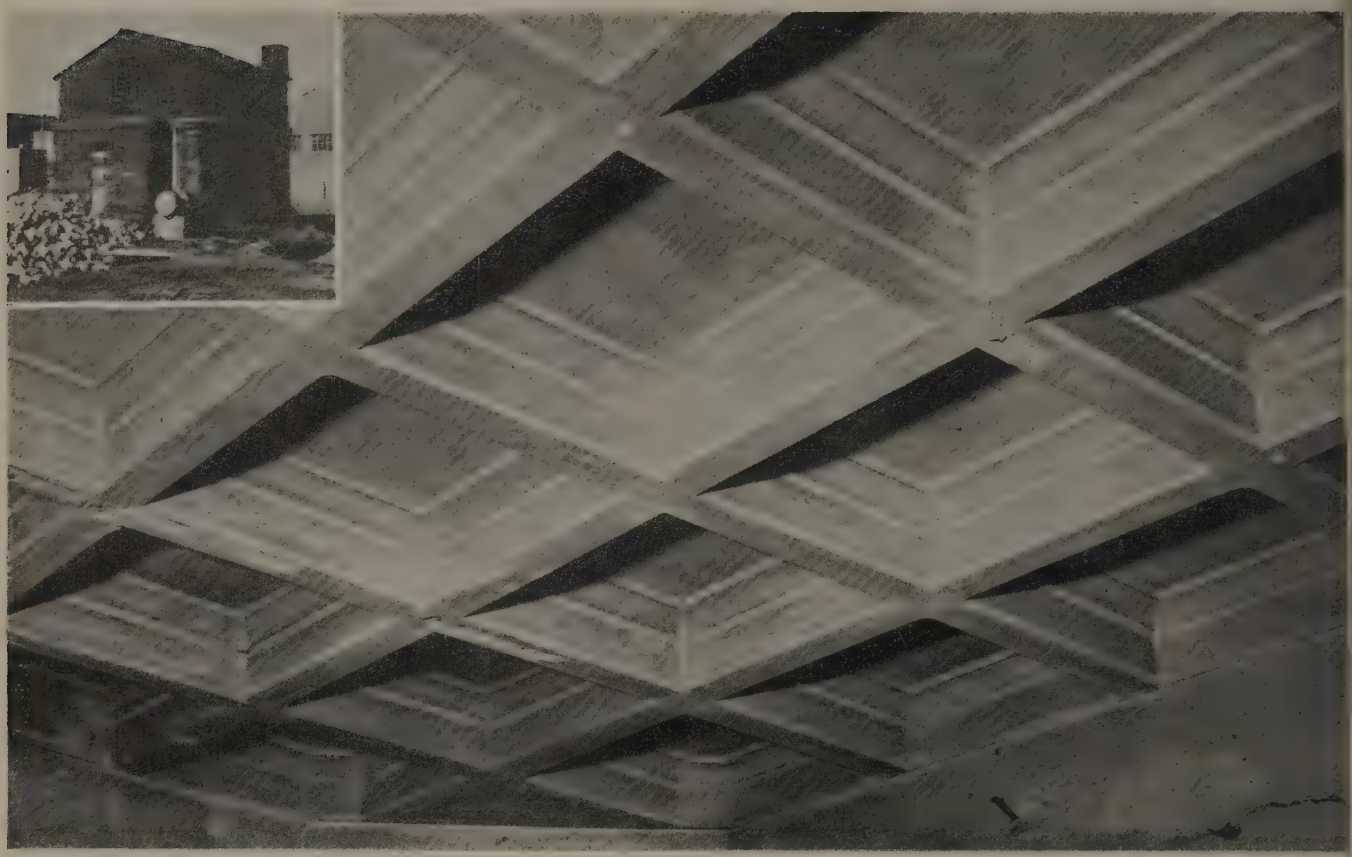
Engineering Contracts and Specifications, 3rd Edition by R. W. Abbett. (New York : John Wiley, 1954, London : Chapman and Hall). 429 plus xv pp., $8\frac{1}{4}$ in. × $5\frac{1}{2}$ in., 48/-.

This is the third edition of an American book which is a basic legal guide to the administration of construction work. The text has been revised and expanded, and brought up to date, without, however, changing the scope and arrangement. The book gives much useful information in a compact form which makes it suitable for reference purposes, and it is also a convenient textbook, the emphasis being on how to write contracts and specifications, rather than on what to write.

(*continued on page 137*)

Early Reinforced Concrete in Newcastle upon Tyne

By Professor W. Fisher Cassie, Ph.D., M.S., F.R.S.E., M.I.C.E., M.I.Struct.E

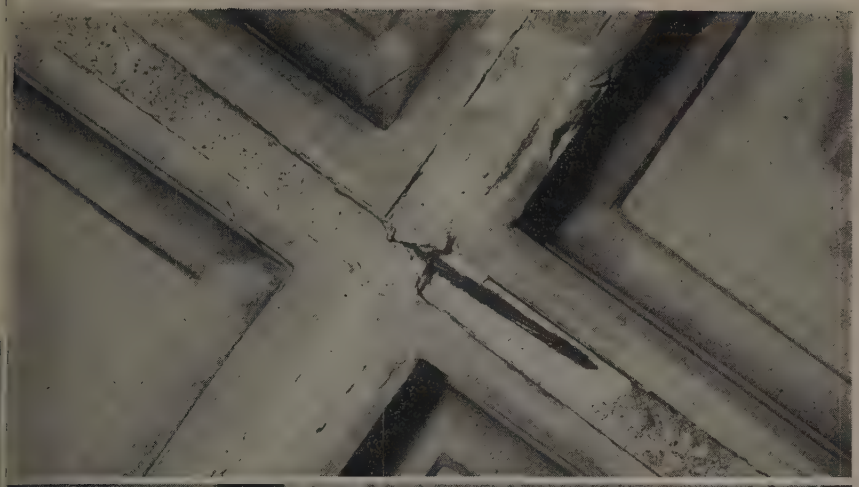


View of the ceiling of the ground floor showing precast plaster panels supported by secondary beams. Inset.—An external view of Wilkinson's house.

THE new laboratories of the Rutherford College of Technology, Newcastle upon Tyne, are now under construction and, to clear the site, it was necessary to demolish a small house which was, at one time, the residence of the foreman of a builder's yard. This house has considerable historical significance in the history of reinforced concrete and the opportunity was taken to investigate its construction.

The house was built under Wilkinson's patent which was taken out on the 27th October, 1854. William Boutland Wilkinson was a Newcastle plasterer who was associated not only with W. Ferguson, whose business still flourishes in Newcastle upon Tyne, but also with William Aspdin whose discovery of Portland cement is well known, and who set up a cement factory in Gateshead about 1852. Wilkinson not only was the first to suggest the curing of concrete by using a layer of wet sand, but was also clearly one of the first to think of and to produce structural reinforced concrete. Wilkinson's patent describes the reinforcing of concrete with flat bars or with second-hand wire ropes from collieries. He points out tha the wire ropes should be inserted where the tension i to be expected, and indicates clearly that he understoo the distribution of tension in a beam. It was nc until a year after Wilkinson's patent that Lambc showed the famous reinforced concrete boat at th Universal Fair in 1855.

The house in question which is shown in the photc graphs, and which was built about 1865, was in excellen condition when it was demolished. It was buil entirely of reinforced concrete including the stairs an the chimney. The walls were 9 in. thick on the firs storey, 12 in. on the ground floor and thickened t 18 in. in lieu of foundations. The walls were dar grey in colour and in appearance resembled moder no-fines concrete. They contained plums of variou shapes and sizes. The most interesting part of th building was the first floor which consisted of precas plaster moulds which had quite obviously been sup ported in position on timbering while Wilkinson' old colliery ropes were laid between the moulds in



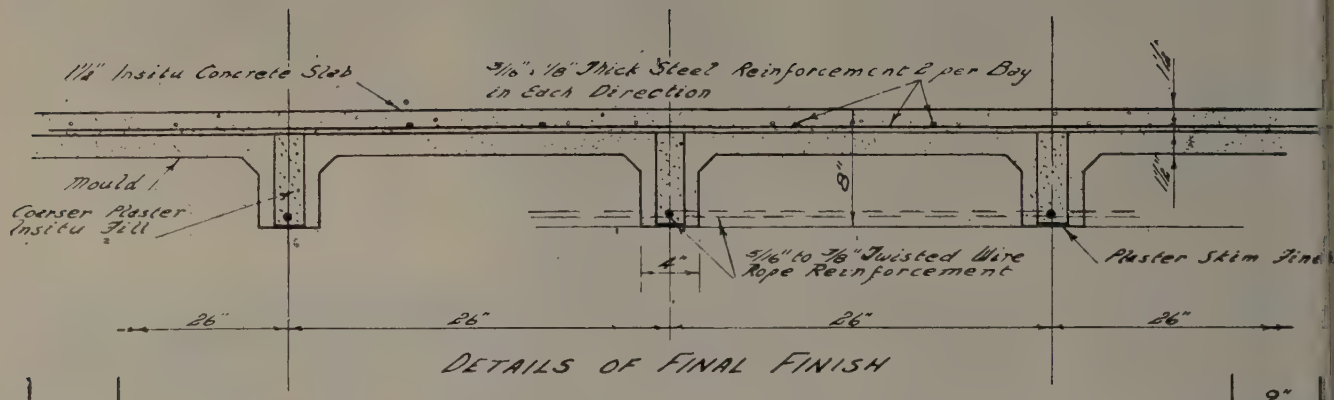
Looking vertically upwards to the ceiling of ground floor showing arrangement of reinforcement between precast plaster panels.

Section cut through the ceiling of the ground floor showing plaster panels with concrete screed above and square mesh of reinforcement.

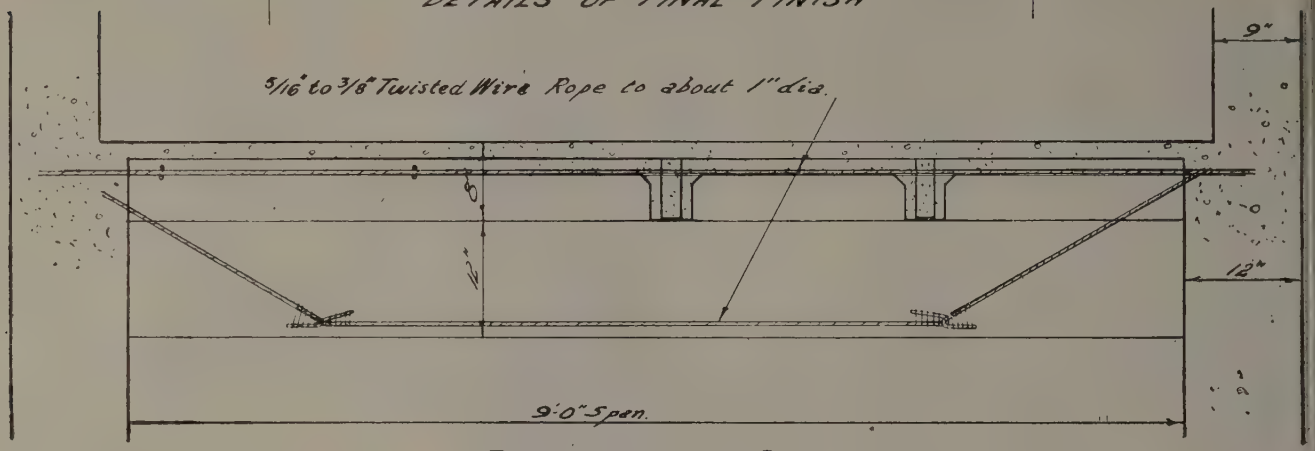
Section cut through main beam showing how wire rope reinforcement was joined and how it was inclined upwards at points where the bending changed sign.

rectangular pattern. The spaces between the moulds had then been filled with coarser plaster *in situ* and with concrete and covered by a concrete slab having a granolithic surface. Beams in this floor were approximately 26 in. apart and were about $6\frac{1}{2}$ in. deep. The reinforcement of these beams was $\frac{5}{16}$ to $\frac{3}{8}$ in. twisted wire rope placed in the bottom but sloping up approximately 30° at the ends before being anchored in the larger beams or in the wall. The plaster *in-situ* filling between the plaster moulds had effectively protected the reinforcement for 90 years. The concrete slab which overlay the hollow tile ceiling and beams was $1\frac{1}{2}$ in. thick and contained $\frac{3}{16} \times \frac{3}{8}$ in. thick steel flats in the bottom running in both directions. Here again the material was unrusted and the span was approximately 12 ft. in each direction.

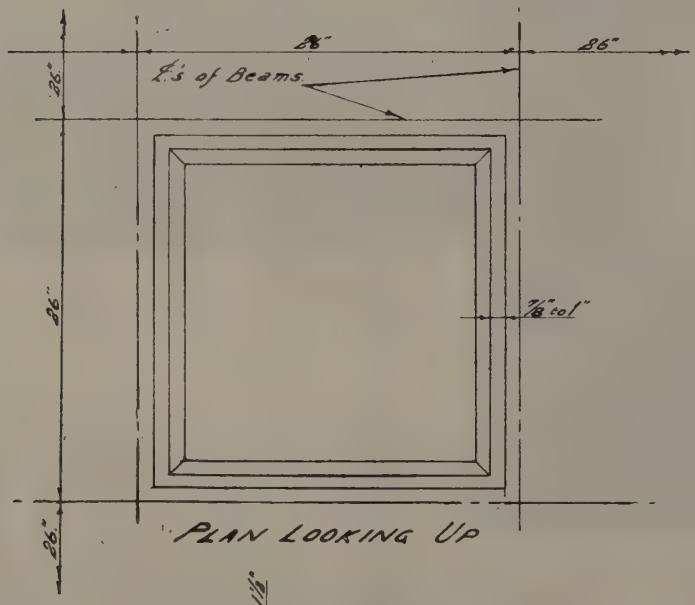
One of the interesting features of this building was a large beam 9 ft. clear span and 9 × 12 in. in cross section made of concrete and reinforced by 1 in. dia. wire rope which had apparently been made up of the smaller ropes used in the slab beams. This beam had been cast in two parts. The first was 12 in. deep to the underside of the slab beams, and then the top portion of 8 in. had been cast with the $1\frac{1}{2}$ in. slab. The wire rope reinforcement was in three parts, the centre portion being horizontal and the two end portions sloping up to the supports. The junction of the pieces was effected by a simple looping of the rope which was then bound with iron wire about $\frac{1}{2}\frac{1}{4}$ in. dia. There was another wire rope in the top of the beam to act as top reinforcement. This was horizontal, in one length and was also used to anchor the slab and beam



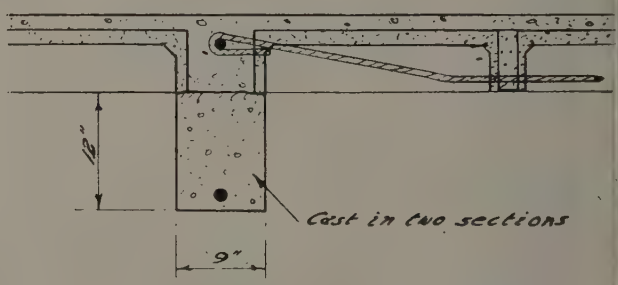
DETAILS OF FINAL FINISH



ELEVATION OF BEAM

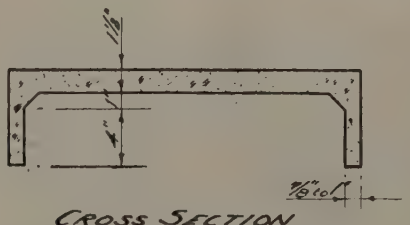


PLAN LOOKING UP



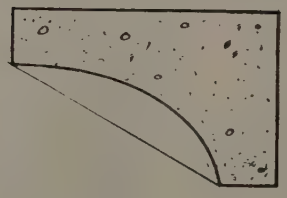
'CROSS SECTION THRO' BEAM

DETAILS OF BEAM OVER BAY



CROSS SECTION

DETAILS OF PRECAST PLASTER SHUTTERING



SKETCH CROSS SECTION OF PRECAST STAIR

Details of First Floor of old concrete house.

Outside appearance of Wilkinson's building showing window and beyond the staircase to the first floor. Every part of the construction was in concrete.

reinforcement, which passed over it and was bent back. The photographs and drawing show the arrangements of these beams and slabs.

A study of the concrete itself was made by carrying out compression tests and examining the material for loss on ignition, for organic matter, and for its geological origin. On 2 in. cubes sawn from the floor, the average strength in compression was 3,800 lb./in^2, the maximum strength 4,500 lb./in^2, and the density 109 lb./ft^3. The aggregate was a hard iron clinker, possibly obtained locally from an iron furnace. Other minerals present were quartz, pyrites and calcium-aluminium silicates. There were indications that the cement was originally very coarse compared with present-day standards. Although organic matter was only 2% the loss on ignition was 37%. The absorption of water after 24 hours' immersion was 27.3%.

It is regrettable that this interesting dwelling had to be demolished, for although Coignet encased structural iron members in 1852 and Lambot showed his boat in 1855, a study of the dates suggests that Wilkinson was the first to reinforce concrete structurally, a distinct advance in ideas and achievement over the reinforcing of light articles by mesh. The recently mounted exhibition by the Cement and Concrete Association offers no hint of this Tyneside development of 100 years ago, and it is hoped that these notes will help to give Wilkinson his proper place in the ranks of the "fathers" of reinforced concrete.

The writer is the reporter for a number of interested engineers and architects who were concerned with the demolition, and he is particularly grateful to the contractors, Messrs. Stephen Easten, to the architect to the Newcastle Education Committee, Mr. Fred Harvey, and to Mr. Tom Bryce and Mr. W. J. Henderson, both of Messrs. Mouchel and Partners for valuable information and assistance. Mr. Clegg of the Department of Photography, King's College, University of Durham, took the photographs. Mr. E. Miller, Dr. P. L. Robinson and Professor T. S. Westoll carried out the physical examination of the concrete.

Book Reviews

(*continued from page 133*)

Municipal Engineering, Administration and Organisation by Rodney S. Offord, B.Sc., A.M.I.C.E., M.I.Mun.E. (London: Contractor's Record Ltd., 1953.) Price 25s.

This book of over two hundred pages, has arrived at a most opportune time and, is full of most interesting details connected with the work of the Municipal Engineer.

In the preface, the Author states that "the older Municipal Engineer will have perfected his own administrative organisation." I disagree with this statement, as no administrative organisation is perfect, and no Municipal Engineer can afford to be complacent. He should be continually alive to the need of adjusting his organisation to rapidly changing circumstances. Ratepayers are entitled to have an efficient economical administration, and the Municipal Engineer, old and young, can profit considerably by following through this exceedingly well written and easily read work.

The first chapter deals with Municipal Engineering and Government, and the succeeding four chapters deal with relationship of officers and workmen with the Council, Departmental Organisation, Works Organisation, and Finance and Economics.

The next five chapters deal with sectional organisation covering the whole field of Municipal Engineering and, finally, there is a peep into the future of Local Government.

There are two appendices. Appendix 'A' covers thirty-three pages of clearly drawn diagrams showing layouts and departmental structures. Appendix 'B,' covering eight pages, is particularly useful in that the principal statutes affecting the Municipal Engineer are listed.

Mr. Offord's book will be of inestimable value to the student and young engineer. The experienced engineer can read these pages with profit, and members of local Councils would get a far greater insight into the work of a Local Authority by a perusal of its pages.

R. T.

F.B.I. Register of British Manufacturers, 26th Edition. (London: Kelly's Directories Ltd., and Iliffe and Sons Ltd., 1954). 952 pp., 9½ in. × 7½ in., 42/-.

This authorised directory of the Federation of British Industries is recognised as a standard export reference book to British Industry. There are seven sections each marked with a thumb index for easy reference. A buyers' guide classifies over 6,500 F.B.I. member firms under more than 5,000 alphabetical trade headings, while full details of all these firms are given in a comprehensive alphabetical directory. Other sections provide useful information about Trade Associations, proprietary names, trade marks, etc.

Institution Notices and Proceedings

IMPORTANT NOTICE TO SUBSCRIBERS

Subscribers, Newsagents and Subscription Agencies at home and abroad are asked to note that as from the 1st January, 1955, copies of THE STRUCTURAL ENGINEER can only be obtained on application to the Secretary of the Institution of Structural Engineers, 11, Upper Belgrave Street, London, S.W.1.

Subscription Rate

The Annual Subscription, post free, to any part of the world is 44s. 6d.

Single Copies

The price of single copies is 3s. 6d. or 3s. 9d. if sent by post.

In order to fulfil all demands, subscribers and others are requested to state their requirements as soon as possible.

The usual rate of discount will apply.

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1., on Thursday, 24th February, 1955, at 5.15 p.m., Dr. S. B. Hamilton, M.Sc., B.Sc.(Eng.), A.R.C.S., M.I.C.E., M.I.Struct.E. (President) in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated, below, should be referred to when consulting the Year Book for evidence of membership.

Students

Birdi, Inder Jeet Singh, of London.
Bratt, James Geoffrey, of Johannesburg, South Africa.
Bucher, Peter Emil, of London.
Choo Yut Shing, of Singapore.
Fong Weng Meng, of Singapore.
Hutchinson, John Lawrence, of Heanor, Derbyshire.
Knight, David Raymond, of Littleover, Derby.
Kwasny, Zdzislaw Wieslaw, of London.
Larrea-Perez, Julian, of Loughborough, Leics.
McElhone, John Patrick, of Palmerston North, New Zealand.
Narozny, Leszek Franciszek, of London.
Saiz-Amigo, Oscar, of Loughborough, Leics.
Shalders, Michael Bloy, of Enfield, Middlesex.
Singh, Parmatma, of Satnampura, Phagwara, (P.E.P.S.U.) India.
Smith, James Allan Slaney, of Cardiff, Glam.
Stuart-William, Bertram Albert Gwyther, of London.

Graduates

Burch, John Percy Copper, B.Sc.(Eng.), London, of Southsea, Hants.
Burke, Terence, of Harrogate, Yorks.
Carter, Anthony Gilbert, of Kemerton, Nr. Tewkesbury, Glos.
Emmett, Robert, B.Sc.Tech. Manchester, of Prestwich, Lancs.
James, Glyn Evans, B.A. Oxford, of Castleton, Nr. Cardiff, Glam.
Mak Wan-Kwong, B.Sc.(Eng.) Hong Kong, D.I.C., of Ashford, Middlesex.
Muscat-Fenech, George, B.Sc., B.A. & E. Malta, Valletta, Malta.
Nankivell, William Hugh, A.R.I.B.A., of Bisho Waltham, Hants.
Noble, John Michael, of Denby Dale, Nr. Hudder field, Yorks.
Orton, Michael Francis, of Wednesbury, Staffs.
Qureshi, Mohd. Arif, of Multan Cantt, (Punja Pakistan.
Rudman, Kenneth Cleve, of Swindon, Wilts.
Slack, John Hartley, of Coventry, Warwicks.
Weston, Raymond John William, of Morden, Surre

Associate-Member

Dickson, Hubert, of Manchester.

Member

Atkins, William Sydney Albert, B.Sc.(Eng.) Londo M.I.C.E., of Lower Kingswood, Surrey.

Transfers

Students to Graduates

Brett, Peter Ronald, of Reigate, Surrey.
Brown, Leslie Arthur, of Trowell, Notts.
Cooke, Brian Herbert, of London.
Lempriere, John Victor, of Middlesbrough, York
Parry, Robert Tudor, of Wirral, Cheshire.
Tomlinson, Thomas Herbert, B.Sc.(Civil) Wales, Westcliff-on-Sea, Essex.

Graduates to Associate-Members

Bonnett, Clifford Frederick, of East Barnet, Hert
Bryant, Michael Ernest, of Beckenham, Kent.
Heath, Roy Edward, of Barnehurst, Kent.

Associate-Members to Members

Agnew, Sydney, of Stretford, Lancs.
Botteley, Gerald, A.M.I.C.E., of Sutton Coldfiel Warwickshire.
Chaffer, John Russell, of Sutton Coldfield, Warwic shire.
Clark, Bernard Leonard, of Sanderstead, Surrey.

Associate-Members to Retired Associate-Membership

Bamford, Wilfred Clifford, of Kells, Co. Meath Ireland.
Brook, Alec Edward, of Ottery St. Mary, Devon.
Leys, Herbert Henry, M.B.E., B.Sc.(Eng.), of Harro Weald, Middlesex.

Members to Retired Members

Grisenthwaite, Thomas Carlyle, B.Sc.(Eng.), M.I.C.E of Northampton.
Hanby, George Ashley, of Hull, Yorks.
James, Edward Marmaduke, M.I.C.E., of Cullercoats Northumberland.
Robinson, George Allen, B.Sc.(Eng.), M.I.Mech.E of London.
Slater, Charles Howard, of Malahide, Co. Dublin
Sturdee, Thomas Percival, of Thames Ditton, Surrey

OBITUARY

The Council regret to announce the deaths of Bertram Blaney CLARKE, Herbert Cecil DAVIS, George FRASER, James HUGILL, John Brian KERSHAW, Donald McGregor NEWTON (Members); Frank IVINSON (Retired Member); Maurice Albion OCKENDEN (Associate); Kenneth Lionel Claude FREEBORN (Associate-Member); and Raymond Francis COLDMAN (Graduate).

RESIGNATIONS

Notification was given that the Council had accepted with regret the resignations of Joseph ADDISON, M.C., Sidney Francis BESTOW, Harold Hodgkinson BROUGHTON, Gordon WELCH (Members); Eric S. BENSON, John Paul Cameron HOUSTON (Associates); Daniel Johannes Jacobus BEZUIDENHOUT, Major Percival Benton BRYDEN, Charles Alfred HART, Cecil William VENTON (Associate-Members); Roger John AXTELL, Charles Joseph BUSHELL, Eric William CLARK, Brian George INGLIS, Richard KIMELMAN, John George MILLS, Arthur David ROGERS, Roy TAYLOR (Graduates); George GIBSON, Edwin Mathieson HAYNES (Students).

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, April 28th, 1955

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. G. P. Bridges, M.I.Struct.E., A.M.I.C.E., L.R.I.B.A., will give a paper on "Colliery Structures."

Thursday, May 26th, 1955

Ordinary General Meeting for the election of members.

Annual General Meeting of the Institution.

Annual General Meeting of the Voting Contributors to the Institution of Structural Engineers' Benevolent Fund.

Members wishing to bring guests to the Ordinary Meeting are requested to apply to the Secretary for tickets of admission.

Thursday, June 23rd, 1955

Ordinary General Meeting, for the election of members.

CANCELLATION OF MEETING

As members in the United Kingdom were notified, the Joint Meeting with the Cement and Concrete Association and the Reinforced Concrete Association arranged for the 24th February at the Central Hall, Westminster, was cancelled owing to the illness of Professor Pier Luigi Nervi, who was to have lectured on "Some Reinforced Concrete Structures in Italy."

EXAMINATIONS, JULY, 1955

The Examinations of the Institution will next be held at centres in the United Kingdom and Overseas on the 12th and 13th July, 1955, (Graduateship) and the 14th and 15th July (Associate-Membership).

REPRESENTATION

The Council have made the following nominations of members to represent the Institution :—

Architects Registration Council and Admission Committee

Mr. F. R. Bullen (Reappointed).

British Standards Institution Committee ISE/- Iron and Steel Industries

Mr. John Mason.

British Standards Institution Committee USM/2/4, Part 4, Strength of Materials and Structures

Dr. E. H. Bateman.

DRURY MEDAL AWARD

The fifth competition for the above award will take place in 1955. The subject is the design of a mobile crane.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary : envelopes to be marked in the top left-hand corner, "Drury Medal Award."

The closing date for the competition is October 1st, 1955.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say, primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution, or failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate in the presence of a Justice of the Peace or a Commissioner for Oaths.

JOURNAL CASES AND BINDING, 1954

A binding case can be supplied for the twelve issues of the Journal, January-December, 1954 (Volume 32), price 11s. 6d., post free. The price for binding in half-leather, inclusive of packing and postage, is 27s. per volume.

It is requested that all parcels containing Journals forwarded for binding should bear the name and address of the sender and must be despatched to reach the Institution by the 4th April, 1955.

An Index will be included in all bound volumes. Members making their own arrangements for binding may obtain a copy of the Index upon application to the Secretary.

Copies of the Index will be provided for subscribers and those on the complimentary list.

INSTITUTION AWARDS

The following awards have been made for the Session 1953-54 :—

Bronze Sessional Medals and London (Headquarters) Prize

Mr. J. S. Terrington and Dr. J. M. Hawkes for their paper on "Crane Gantry Girders for Steelworks."

Bronze Research Medals

Dr. R. H. Wood, for a paper on "A Derivation of Maximum Stanchion Moments in Multi-storey Frames by means of Nomograms."

Dr. P. B. Morice and Mr. G. Little for a paper on "Load Distribution in Prestressed Concrete Bridge Systems."

Lancashire and Cheshire Branch Prize

Professor J. A. L. Matheson, for the Chairman's Address.

Midland Counties Branch Prize
Mr. H. V. Hill, for a paper on "The Load Bearing Capacity of Metal Structures."

Northern Counties Branch Prize
Professor W. Fisher Cassie, for a paper on "Pavement Structures."

Northern Ireland Branch Prize
Mr. R. J. N. Sweetnam, for a paper on "Reconstruction of Queen's Quay, Belfast Harbour."

South Western Counties Branch Prize
Mr. C. J. Woodrow, for a paper on "The Tamar Bridge Proposal."

Wales and Monmouthshire Branch Prize
Dr. A. A. Fordham, for a paper on "Research Work in Soil Mechanics at the University College of Swansea."

Western Counties Branch Prize
Mr. J. E. Collins, for a paper on "The Industrial Architect and Engineer."

Yorkshire Branch Prize
Mr. E. Lightfoot, for a paper on "Dynamic Stresses in Structures."

LONDON GRADUATES' AND STUDENTS' SECTION

Hon. Secretary: J. A. Pope, 53, Cranleigh Drive, Leigh-on-Sea, Essex.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The Annual Business Meeting of the Branch will be held at the College of Technology, Manchester, on Thursday, 28th April, 1955, at 6.30 p.m., and will be followed by a film.

Joint Hon. Secretary: A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire; M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford, 6, Lancs.

MIDLAND COUNTIES BRANCH

The Annual General Meeting of the Branch will be held at the James Watt Memorial Institute, Birmingham, on Friday, 22nd April, 1955, at 6 p.m., and will be followed by a Joint Meeting with the Reinforced Concrete Association, when Mr. J. A. Derrington, B.Sc., D.I.C., A.M.I.C.E., A.M.I.Struct.E., will give a paper on "Recent Developments in the Technique of Precasting Concrete Structures."

A meeting of the Branch will be held at the Gas Showrooms, Nottingham, on Tuesday, 26th April, 1955, at 7 p.m., when Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E., M.I.Struct.E., M.Inst.W.E., M.Am.Soc.C.E., M.Cons.E. (Hon. Librarian) will give a paper on "Some Unusual Industrial Structures."
Hon. Secretary: L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES GRADUATES' AND STUDENTS' SECTION

Hon. Secretary: A. K. A. Costain, A.M.I.C.E., 134, Witherford Way, Weoley Hill, Birmingham, 29.

NORTHERN COUNTIES BRANCH

The Annual General Meeting of the Branch will be held at the Cleveland Scientific and Technical Institution, Middlesbrough, on Tuesday, 5th April, 1955, at 6.30 p.m., when Mr. D. J. Leggot, A.M.I.C.E., A.M.I.Struct.E. will give a paper on "Some Aspects of Industrial Building Design."

Hon. Secretary: Captain O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

Hon. Secretary: A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., "Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The Annual General Meeting of the Branch will be held at the Cadoro Restaurant, Glasgow, on Tuesday, 26th April, 1955, at 6 p.m.

Hon. Secretary: G. Drysdale, A.M.I.Struct.E., "Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH WESTERN COUNTIES BRANCH

The Annual General Meeting of the Branch will be held at the Duke of Cornwall Hotel, Plymouth, on Friday, 20th May, 1955, at 7 p.m.

Joint Hon. Secretaries: E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay; C. J. Woodrow, "Elstow," Hartley Park Villas, Tavistock Road, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The Annual Dinner will be held at Porthcawl on Friday, 22nd April, 1955.

The Annual General Meeting of the Branch will be held at the offices of the Steel Company of Wales, Port Talbot, on Tuesday, 3rd May, 1955, at 6.30 p.m.
Hon. Secretary: K. J. Stewart, A.M.I.C.E., A.M.I.Struct.E., 15, Glanmor Road, Swansea.

WESTERN COUNTIES BRANCH

The Annual General Meeting of the Branch will be held at the University of Bristol Geology Lecture Theatre, on Friday, 1st April, 1955, at 6 p.m., and will be followed by a film show.

Hon. Secretary: E. Hughes, M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The Annual General Meeting of the Branch will be held at the Great Northern Hotel, Leeds, on Wednesday, 20th April, 1955, at 6.30 p.m., and will be followed by a paper.

Hon. Secretary: E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary: A. E. Tait, B.Sc., A.M.I.C.E., P.O. Box No. 3306, Johannesburg, South Africa.

During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone: 34-1111, Ext. 257.

Natal Hon. Secretary: E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co. Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary: R. Stubbs, M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

Report on Structural Safety

Note. This report has been prepared by a special committee of the Institution of Structural Engineers and authorized by the Council for publication with written discussion. The Report will also be presented at an Ordinary Meeting of the Institution to be held on Thursday, 24th November, 1955, and it is requested that written comments be forwarded to the Secretary of the Institution not later than the 14th September, 1955.

Foreword

This report has been compiled by a committee of the Institution after a review of safety problems in relation to a wide range of structures. In its report the committee makes a fresh approach to structural safety problems with a view to helping the development of more rational methods of design. No attempt has been made to put forward recommended numerical values for load factors applicable to the various mediums of construction, this being considered to be a matter for Codes of Practice committees and others working in the respective fields.

The report should not be regarded as final and is published in this Journal in order to secure wide publication in a way that will bring about appropriate comment and discussion. Written discussion is invited and it is hoped thereby to secure the views of many research workers and others interested in this subject. All such contributions will be carefully considered by the committee with a view to the preparation of a final report on the broadest possible basis.

The committee has in mind that such a report when prepared may prove to be a first step towards the achievement of a Code of Practice for a more rational approach to design generally applicable to all types of structures and materials.

1. Introduction

The committee was appointed in December, 1951, with the following terms of reference :—

(*a*) To review structural safety problems, including modes of specifying margins of safety for design purposes.

(*b*) To prepare a report for discussion and for publication.

The committee has met 20 times between then and October, 1954, and this report sets out its conclusions in a form thought suitable for written discussion.

The report seeks to advance current ideas regarding structural safety and to provide a basis for the more rational formulation of safety clauses in future revised or new codes of practice.

The committee has not attempted to compile a comprehensive bibliography on structural safety problems, but in Appendix V gives some valuable general references wherein may be found useful lists of relevant papers.

2. Basic Requirements

Whatever the degree of safety of a given structure or the mode of its specification, some basic needs must, directly or indirectly, be satisfied. The following requirements are put forward as expressing the fundamentals of safe construction, each of which, for a given structure, should be satisfied to a reasonable degree of probability :—

(*a*) That the structure shall retain, throughout its life, the characteristics essential for fulfilling adequately the purpose for which it was constructed, without abnormal maintenance cost.

(*b*) That the structure shall retain throughout its life an appearance not disquieting to the user and general public, and shall neither have nor develop characteristics leading to concern as to structural safety.

(*c*) That the structure shall be so designed that adequate warning of danger is given by visible signs ; and that none of these signs shall be evident under design working loads.

In the application of these three basic requirements to a given case, it is envisaged that the type of construction and the proposed life of the structure shall be decided by the owner, advised by the engineer and having due regard to the structural and use limitations expressed by him. To prevent a structure being misused after erection, attention is drawn to the value of regular inspection, of displaying suitable loading restriction notices, and of arranging for the notification to responsible authorities of change of use of the structure. Consideration should be given to the safe extensions of life that can result from good maintenance coupled with arrangements of this sort.

Typical of the warning signs envisaged in (*c*) above are the development of abnormal deflections, marked local deformation or cracks, or observable permanent set. The more evident the warning signs, the greater will be the chance of detecting and remedying defects, and hence preventing collapse.

3. Design Information

The information on which the safety of a proposed structure must be assessed will, in general, comprise :—

(*a*) The site and foundation conditions.
(*b*) The materials to be used and the standard of workmanship.
(*c*) Particulars of the design and of the loading conditions.
(*d*) Design calculations and relevant tests.

Whatever the merits of the design calculations, safety can be advanced by experimental information on the behaviour under load of both the materials to be used and of the structure itself, in model or other form.

With regard to materials, the primary design need is a decision on a failing strength value for each material used. For this purpose measurements of strength should be made to determine both the average strength (m) of the material and also the variation (expressed by the standard deviation σ) of its strength, natural to the production process, about the average. The design failing stress should then be based upon the average strength less an amount depending upon the variation of the strength chosen to correspond to some given small risk of material of strength less than the design value occurring in the actual structure. Thus, for example, the design stress value could be taken

as $(m—k\sigma)$, with k equal to 2.3, thereby involving, for a Gaussian distribution of the test values, a risk that 1 per cent. of the actual material will have its strength below the design value. In choosing a design failing stress, extreme faults (such as large scale foreign matter in concrete) that can be avoided by ordinary controls and inspection methods should be neglected. In any event, control and inspection of materials should be organised in the light of the considerations that went to the decision on the design failing stress value.

With regard to structural tests, it is believed that safety is most economically advanced by tests on ad hoc models or by tests on representative structures, model or full scale, for research purposes. Deflection tests, proof tests and tests to destruction also have their value where practicable. It is recommended that regulations affecting the safety of structures should always provide for the admission and use of experimental evidence as to the behaviour of a structure or its components under load.

The main body of evidence regarding the safety of a structure, however, will usually take the form of design calculations. It is assumed throughout this report that these calculations will have standards of accuracy and completeness at least equal to the standards set by the relevant current codes of practice. It is realised, however, that the state of knowledge regarding different types of construction is never, at a given moment and for a given designer, such as to ensure a uniform standard of safety, and provision is made, in a later section of this report, whereby margins of safety may be varied to allow for these inequalities. As a guide to judgment in this matter, the ideal procedure is regarded as comprising full calculations relating to the behaviour of the structure under both working load and collapse load conditions, using material properties and loading data based on the statistical analysis of extensive tests and operational observations respectively. Due allowances for any shrinkage and temperature effects, for the dynamic effects of live loads, for corrosion and for changes in support conditions are assumed. The calculations contemplated provide for the treatment of any redundancies or breakdowns of linear response to load that may arise, the treatment to be as full and accurate as that found practicable at the time by leading specialists in the profession.

4. Margins of Safety

In discussing margins of safety for structures, it has been thought best to concentrate on load ratios rather than stress ratios or working stresses. This decision accords with most modern concepts of structural safety. Two kinds of ratio are considered :—

(*a*) The ratio of the ultimate load to the appropriate working load, known as the ULTIMATE LOAD FACTOR.

(*b*) The ratio of the limiting load to the appropriate working load, known as the LIMITING LOAD FACTOR.

The actual loading on a given structure will in general vary during its life in both magnitude and distribution. For a given class of structure and of user, such variations can be studied statistically and thence, within given safety limits, represented for design purposes by one or more specified working loads, each of given magnitude and distribution. In doing this, and hereafter in this report, a working load is taken, for a particular distribution, as the greatest loading that has a reasonable chance of occurring on the structure during its life while used in the manner for which it was designed. A limiting load is the loading, distributed in the same way as the working load, at which some limiting condition such as excessive deflections or cracking, first arises. An ultimate load (often termed the collapse load) is the maximum load, distributed in the same way as the working load, that the structure can carry.

Of the two ratios in (*a*) and (*b*) above, the ultimate load factor is regarded as of primary importance, and somewhat more attention is therefore given to it in the remainder of this report than to the limiting load factor.

It is realised that safety cannot be ensured by calculations relating to the ultimate load factor and limiting load factor alone, and that factors such as fatigue and vibration, instability due to aerodynamic causes and fire damage, must receive separate and proper consideration over and above any incidental provision that may be made against collapse on the counts during the course of ordinary design calculations. Such further considerations, however, are deemed beyond the scope of this report.

5. Considerations affecting the choice of ultimate load factors

The selection of the ultimate load factor to be used in the design of a given structure, though commonly made empirically in the light of experience, must fundamentally be based upon the assessment of a range of considerations that can be conveniently grouped as follows :—

Group X. Factors influencing the probability of collapse

A. Workmanship, having regard to inspection, maintenance and materials.
B. Loading, having regard to control of use.
C. Accuracy of analysis, having regard to type of structure.

Group Y. Factors influencing the seriousness of the results of collapse

D. Danger to personnel.
E. Economic considerations.

It will be seen that the five headings given, fall naturally into two quite different groups, the first of which has always been explicitly considered while the second has hitherto had its influence largely indirectly and often without acknowledgement.

Each of the five headings is amplified and defined below. In the first three cases, *A*, *B* and *C*, this has been done by reference in each case to optimum conditions and adverse factors ; in this way, an effort has been made to help in the assessment of a given structure in relation to the parameters concerned.

A. Workmanship, having regard to inspection, maintenance and materials

Optimum conditions

(*a*) Work executed by skilled craftsmen under efficient supervision in good fabrication shop conditions and erected in reasonably clement weather.
(*b*) Inspection and control of materials in accordance with up-to-date Codes of Practice.
(*c*) Effective maintenance ensured by good design, periodic inspection and limitation of corrosion risk.

(*d*) Adequate statistical data on strength of materials; materials having good ductility and fatigue resistance.

Adverse factors

(*a*) Inadequately skilled labour or poor supervision.
(*b*) Construction necessarily carried out in inclement weather, or with undue haste.
(*c*) Little skilled inspection or means of repair.
(*d*) Inaccessibility of the structure or parts thereof for inspection and maintenance, due to position of structure or to difficulties inherent in the design.
(*e*) Corrosion losses during the life of the structure likely to be large or not predictable.
(*f*) Inadequate data on strength and other relevant material properties, lack of ductility.

B. Loading, having regard to control of use

Optimum conditions

(*a*) Working load naturally well defined or based on statistical surveys.
(*b*) Control of use—overloading prevented by some positive means, natural or otherwise; change of user or nature of use unlikely or well controlled.

Adverse factors

(*a*) Absence of well defined load or adequate statistical data on loading.
(*b*) Use experimental, liable to change, unpredictable or uncontrolled.
(*c*) Loading highly variable, in time (rapid or slow) and in distribution, to uncertain extents. Maximum loads likely to exist for unusually long periods of time.

C. Accuracy of analysis, having regard to type of structure

Optimum conditions

(*a*) Calculations based on reliable material properties and adequate loading data.
(*b*) Loading of steady nature, free from dynamic actions.
(*c*) Calculations aimed at direct prediction of ultimate strength of structure, under deformation conditions fully appropriate to the ultimate load, and accurate to within a few per cent.
(*d*) Structure redundant so that partial failure (readily observable) does not result in complete collapse.
(*e*) Structure such that warning signs are given before even partial failure can occur.
(*f*) Structure such that it has no really critical components (critical in the sense that individual failure would precipitate wholesale collapse as might occur with the columns of a tower).
(*g*) Structure with its foundations and other supports unlikely to move.

Adverse factors

(*a*) Material properties and loading uncertain.
(*b*) Loading effects, due to dynamic actions, difficult to calculate.
(*c*) Calculations such as to give only indirect guidance to ultimate strength (e.g. methods of a rule of thumb nature).
(*d*) Structure statically determinate and likely to give little warning if collapse is approached.
(*e*) Structure such that its strength assessment is for some reason specially uncertain.
(*f*) Structure such that it has numerous critical components.
(*g*) Structure with foundations and supports liable to move.

D. Danger to personnel

By this is meant the danger to life and limb of any reasonable users of the structure and of members of the general public who may be involved in its collapse. To a less extent, consideration is to be given here to the subsequent reaction of potential users and of the general public, leading, for example, to a loss of public confidence in structures of the kind concerned.

E. Economic considerations

The cost in money and time to replace or repair the structure are envisaged here, together with the economic effects of having to make do meanwhile without the use of the structure. The existence or otherwise of alternative ways of meeting the need originally provided for by the structure will affect the seriousness of its loss, temporary or permanent as it may be.

6. Systems for assessing ultimate load factors

It was early appreciated that if the assessment of a structure in terms of the five items listed above could be placed upon a satisfactory quantitative basis, a means might be found whereby an appropriate ultimate load factor could be estimated. A number of typical structures have therefore been assessed independently by each of the members of the committee using both numerical marking (out of ten) and allocation to three or four categories (such as very good, good, fair and poor), and as a result it has been concluded that such assessment, at least in terms of four categories for each of the five items listed in paragraph 5, can be achieved with good agreement among professionally qualified assessors. A number of ways have therefore been explored of using the results of such assessment to estimate the ultimate load factor that is appropriate. The tabular system described here is one favoured by the majority of the committee as being the simplest and most flexible. Some other ways that have been studied by the committee and found of interest are given in Appendices I, II, III, and IV.

It is important to consider the objects and limitations of any such system. The aim has been to present first a system of assessing the main relevant characteristics of a structure under the items of Group *X* and Group *Y* above; and by so doing to produce a method of ensuring that these characteristics are, at the lowest, clearly brought to mind when deciding upon an ultimate load factor, and, at the best, so assessed that they form a basis for the evaluation of the factor itself. Secondly, the system of evaluation is put forward as a rational one bringing out the influence of Group *X* and Group *Y* characteristics upon each other and so contributing to the logical determination of load factors. Finally, as a result of an extensive trial and error process in relation to current load factors derived from experience, it is thought that the system presented has sufficient numerical value to warrant consideration by professional engineers with a view to its use both as a general guide and as a system worthy of further

improvement to meet the need of any particular branch or form of construction.

The Tabular System

A simple system for assessing the ultimate load factor for a given structure is possible using two tables from one of which is deduced an X factor, depending on the items A, B and C affecting probability of collapse, and from the other is deduced a Y factor, depending on the items D and E affecting the seriousness of collapse. The load factor is then the product of the X and Y factors.

Considering first the X factor, each of the groups A, B and C referred to in paragraph 5 is given a certain rating by the designer. The ratings are :—

Very good (vg)
Good (g)
Fair (f)
Poor (p)

A "good" or "very good" rating is adopted when the characteristic under consideration is such that the probability of collapse is relatively small. For example, the loading of a water tank is known within closely defined limits so that the rating for B, "Loading, having regard to control of use," would be vg, very good.

On the other hand, where the circumstances are such that the risk of collapse is relatively high, the rating would be "fair" or "poor." For example, if brickwork is built from bricks of very variable strength and the workmanship is not controlled, the rating for A, "Workmanship, having regard to inspection, maintenance and materials," would be p, poor.

Where the design and construction is in accordance with the appropriate code of practice, the ratings for all three characteristics A, B and C should normally be g, good. At present this does not always apply, because the codes represent different stages of development and are not based on the philosophic approach considered in this report. For example, in dealing with the strength of materials, the basis of design for steel, concrete and brick buildings is :—

For steel, the guaranteed minimum yield stress.

For concrete, the minimum cube strength from works tests on three cubes ; if the difference between the maximum and minimum strengths is not greater than 15 per cent, the average strength of the three cubes can be adopted.

For bricks, the average strength of a sample of twelve bricks.

The use of such alternative methods for determining the strengths to be assumed in design, coupled with the differences in workmanship which are envisaged by the various codes, leads to differing risks of collapse and hence the need for differing load factors.

The ratings for A, "Workmanship, etc.", would be rather better than "good" for steel, rather poorer than "good" for concrete, and "poor" for brickwork. For this last material, half the bricks may be below the mean value assumed in design and an important proportion may have strengths 70 to 80 per cent of the assumed value. The ratings would be more nearly the same for the three materials if their strengths were each assumed in design to be $m—2.3\sigma$, where m denotes the mean strength and σ denotes the standard deviation.

Having assessed the rating of each of the characteristics A, B and C, the X factor is obtained from Table 1. The interrelationship between the factors given in this table conforms to certain logical rules so that the result involves a basic factor of just over unity when circumstances are nearly perfect, also a term to which each of the characteristics contributes individually, and a term involving the results of interaction between the effects of variation of the three characteristics. The two-decimal figures included in this table are to show the ordered progression of the values therein and are not, of course, indicative of a corresponding degree of accuracy. Factors above 2.5 are given in italics. In many cases that relate to such values it may well be that, rather than adopt such factors, it would be better, by adopting statistical methods of load and strength assessment or superior modes of calculation, to alter the practical conditions envisaged.

With regard to the Y factor, this depends on only two characteristics, D denoting "Danger to personnel" and E denoting "Economic considerations" as discussed in paragraph 5. Each of these is assessed in terms of three categories :

Not serious (ns)
Serious (s)
Very serious (vs)

The Y factor is then obtained from Table 2. The Y factor is used essentially to *modify* the factor deduced from the X group and hence has a minimum value of unity. The modifications are on a simple linear basis and the rating of one characteristic is assumed to have no interaction on the rating of the other.

Table 1. Values of X factors

Characteristic		B = vg	B = g	B = f	B = p
A = vg	C = vg	1.1	1.3	1.5	1.7
	C = g	1.2	1.45	1.7	1.95
	C = f	1.3	1.6	1.9	2.2
	C = p	1.4	1.75	2.1	2.45
A = g	C = vg	1.3	1.55	1.8	2.05
	C = g	1.45	1.75	2.05	2.35
	C = f	1.6	1.95	2.3	*2.65*
	C = p	1.75	2.15	*2.55*	*2.95*
A = f	C = vg	1.5	1.8	2.1	2.4
	C = g	1.7	2.05	2.4	*2.75*
	C = f	1.9	2.3	*2.7*	*3.1*
	C = p	2.1	*2.55*	*3.0*	*3.45*
A = p	C = vg	1.7	2.15	2.4	*2.75*
	C = g	1.95	2.35	*2.75*	*3.15*
	C = f	2.2	*2.65*	*3.1*	*3.55*
	C = p	2.45	*2.95*	*3.45*	*3.95*

Table 2. Value of Y factors

Characteristic		D = Not serious	D = Serious	D = Very serious
E =	Not serious	1.0	1.2	1.4
	Serious	1.1	1.3	1.5
	Very serious	1.2	1.4	1.6

Having evaluated the X and Y factors in this way from Tables 1 and 2, the results are combined thus to give the appropriate ultimate load factor :—

Ultimate load factor = (X factor) × (Y factor)

Examples illustrating the use of the Tabular System

(*a*) *Water tank*

Consider a reinforced concrete tank designed for erection on a tower for use as a source of water supply in a country district.

Assessments of items A to E are :—

A = g	D = ns
B = vg	E = s
C = g	

Hence, from Tables 1 and 2, X factor = 1.45, Y factor = 1.1 ; therefore ultimate load factor = 1.60.

(*b*) *Gallery of theatre*

Consider the steel structure of the gallery of a metropolitan theatre. Assessments of items A to E are

A = g	D = vs
B = vg	E = s
C = g	

Hence, from Table 1 and 2, X factor = 1.45, Y factor =1.5 ; therefore ultimate load factor = 2.18.

(*c*) *Walls of a large block of flats*

Consider the load bearing walls to be built of brick under ordinary municipal conditions. Assessments of items A to E are :—

A = p	D = s
B = g	E = vs
C = p	

Hence, from Tables 1 and 2, X factor = 2.95, Y factor = 1.4 ; therefore ultimate load factor = 4.13.

7. Limiting Loads

The margin between the limiting load for a structure and the corresponding working load is commonly much smaller than that between the ultimate load and the working load for the same structure. The definition of limiting load adopted by the committee has already been given in paragraph 4, and it remains to consider the nature of the limiting conditions referred to therein. The conditions envisaged are as follows :—

(*a*) Excessive elastic deflections, limits to which may be set by aesthetic considerations or by some resulting interference with the proper use of the structure.

(*b*) Permanent deflections, with limits set by considerations of the types mentioned in (*a*) above.

(*c*) Development of local defects, such as cracks or local deformations, the limits to which may be set by aesthetic considerations or by effects on corrosion and strength properties.

There are in addition certain other limiting conditions not directly measurable in terms of a limiting load. Of such are excessive vibrations, with perhaps associated human discomfort, unstable oscillations with possible consequent danger of collapse, and damage by fire. In practice, these matters will in part be dealt with by direct treatment independent of the ordinary design use of the concept of a limiting load.

For a given type of construction, and more particularly for a given structure, it will commonly be desirable to specify limits to conditions (*a*), (*b*) and (*c*) above. But such limits, appropriate in a code of practice, will vary considerably with the construction and circumstances of the structure, and the committee has therefore not attempted to attach general numerical values to them.

8. Practical recommendations

As a result of the deliberations of the committee, in addition to and growing out of the foregoing paragraphs of this report, it is desired to draw attention, and in particular the attention of Codes of Practice committees, to the following recommendations affecting current practice :—

(*a*) *Load and material data*

Codes of Practice should be drafted to envisage and encourage an increasing use of the realistic statistical approach, conceding benefits to practices leading to improved knowledge of loads and of material properties.

(*b*) *Test data*

Codes of Practice should be drafted to encourage the use of model and full scale structural tests to improve knowledge of the behaviour of the structure concerned, and to allow, with suitable safeguards, of appeal to relevant test data in the absence of, or to supplement, suitable design calculations.

(*c*) *Ultimate loads*

Codes of Practice should be drafted to encourage the use of calculations and tests directed to the prediction of collapse loads, supplemented by calculations and tests aiming to ensure satisfactory behaviour at lower (working) loads.

(*d*) *Limiting conditions*

Endeavours should be made to specify such conditions in increasing detail in relation to each of the commoner classes of structure.

(*e*) *Design life*

Where practicable, Codes of Practice should give guidance on, and standardise modes of specifying, the life to be expected of the commoner types of structure.

(*f*) *Ultimate load factors*

Ultimate load factors should be decided upon having regard to the probability of collapse, but provision should be made for some variation of ultimate factors according to the seriousness of the practical effects of collapse in any given case.

(*g*) *Further research*

More research in this field is obviously needed and, in particular, the committee recommends work on :—

(i) Loading statistics
(ii) Methods of predicting ultimate strength
(iii) The specification of limiting conditions
(iv) Systems for assessing load factors.

Appendix I

Evaluation of ultimate load factors by marking system

1. *Method*

In this system, each of a number of items under Groups X and Y of the report are given marks out of 10, high marking corresponding to good (= safe and not serious conditions). These marks are then added up to give totals (X and Y) for each group. Then

the ultimate load factor required is determined by the formula

$$N = \frac{\alpha}{X + \beta Y},$$

where α and β are empirical constants.

A development of this method includes provision for relative weighting of the individual items prior to their addition, but this is not elaborated here.

In applying this method, any number of separate items can be listed separately in the Groups X and Y, but for convenience, the items A, B and C of group X and D and E of Group Y, as defined in this report, are used here. In this case the total marks to be awarded have a maximum value of 50. If the relative weighting of the X and Y marks is neglected, β is 1 and the above formula becomes

$$N = \frac{60}{X + Y.}$$

2. *Water tank example*

Tank of reinforced concrete to provide water supply for isolated community. Marks awarded in the X Group are :—

A.	Workmanship, etc.	8
B.	Loading, etc.	10
C.	Accuracy of analysis, etc.	7
	Total = X =	25

Marks awarded in the Y Group are :—

D.	Danger	8
E.	Economy	4
	Total = Y =	12

Hence from the given formula :—

$$\text{Ultimate load factor } N = \frac{60}{25 + 12} = 1.62$$

Appendix II

Evaluation of ultimate load factors by direct weighting system

1. *Method*

In this system, the value of the ultimate load factor for a given structure is determined by assessing the " weight " of the following few parameters and then applying the formula :

$$\text{Ultimate load factor} = \frac{\Sigma W}{20} + 1$$

The parameters considered are :

(1) *Seriousness of results of failure (human or economic)*
Assess weight between 0 (not serious) and 8 (very serious).

(2) *Workmanship*
Assess weight between 0 (very good) and (occasionally bad).

(3) *Load conditions, including consideration given combined loads*
Assess weight between 0 (accurate and co trolled) and 4 (less accurate and uncontrolle

(4) *Importance of the member in the structure*
Assess weight between 0 (unimportant) and (vital).

(5) *Warning of failure*
Assess weight between 0 (ample) and 2 (non

(6) *Depreciation of strength, including provision f maintenance*
Assess weight between 0 (none likely) and (maintenance normal).

2. *Comments on basis of method*

It is assumed that this method will be applied on to structures for which modern forms of ultima load theory are available, and are likely to permit assessments of ultimate strength within about 15 p cent. Due allowances for eccentricities, support mov ments, etc., appropriate to ultimate load condition and provision against fatigue, are assumed. On th basis the ultimate load factors provided for are limite to between 1 and 2. Structures such as load carryin brick walls, which do not meet the above condition and are usually designed with load factors of 3 4, are thus excluded from consideration in this a pendix.

3. *Hangar example*

In giving an example of the method, it has in th appendix been thought of interest to consider a aircraft hangar, not to just one specification but t eight alternative ones, to illustrate the variations ultimate load factor involved. Those eight alte natives are given in Tables 3 and 4.

Table 3.

Altern. No.	Constitution	Site	Use
1	Site reinforced concrete	exposed	public assembly occasionally
2	,, ,, ,,	shel- tered	,, ,, ,,
3	,, ,, ,,	,,	storage non-damageable materia
4	,, ,, ,,	exposed	,, ,, ,,
5	Precast reinforced concrete	,,	public assembly occasionally
6	,, ,, ,,	shel- tered	,, ,, ,,
7	,, ,, ,,	,,	storage non-damageable material
8	,, ,, ,,	exposed	,, ,, ,,

Table 4 below gives the factors derived by the proposed system for the eight alternatives given above

Table 4.

	Parameter	Weights for alternative No. 1	2	3	4	5	6	7	8
(1)	Results failure	8	8	2	2	8	8	2	2
(2)	Workmanship	4	4	4	4	1	1	1	1
(3)	Load conditions	4	2	2	4	4	2	2	4
(4)	Importance of the member in the structure	1	1	1	1	1	1	1	1
(5)	Warning of failure	2	2	2	2	2	2	2	2
(6)	Depreciation of strength	1	1	1	1	1	1	1	1
	ΣW	20	18	12	14	17	15	9	11
	Ultimate load factor $= \frac{\Sigma W}{20} + 1 =$	2	1.9	1.6	1.7	1.85	1.75	1.45	1.55

Appendix III

Evaluation of ultimate load factors by nomographic charts

1. *Method*

This system employs nomograms for the assessment of ultimate load factors in conjunction with a numerical marking for five items affecting the probability of collapse and the seriousness thereof.

In the form of numerical marking adopted, each item is marked out of ten, a high mark corresponding to conditions approaching the optimum. The nomograms for the determination of the X and Y factors are drawn to give results corresponding closely to the values derived by the Tabular System of this report. The influence of each item of the X or Y groups is the same in each system. The product of X and Y giving the load factor is given by a third nomogram.

2. *Theatre gallery example*

Steel structure of a gallery of a metropolitan theatre.

Each of the items A, B and C of X Group in the report are given marks out of 10, high marks corresponding to good (safe) conditions. In this example, the marks awarded are :—

A.	Workmanship, etc.	.. 8
B.	Loading, etc.	 8
C.	Accuracy of analysis, etc.	.. 7

Working from left to right on Chart 1, a straight-edge placed on 7 (the marks for item C) on the first scale with 16 (the sum of marks for items A and B) on the second scale will determine a point on the pivot line. If the straight-edge is turned about this point until it passes through 64 (the product of marks for items A and B) on the last scale, the X factor may be read off the penultimate scale as 1.61. Similarly, the items D and E of Y Group in the report are given marks out of 10, high marks corresponding to good (not serious) conditions. In this example, the marks awarded are :—

D.	Danger	 2
E.	Economy	 2

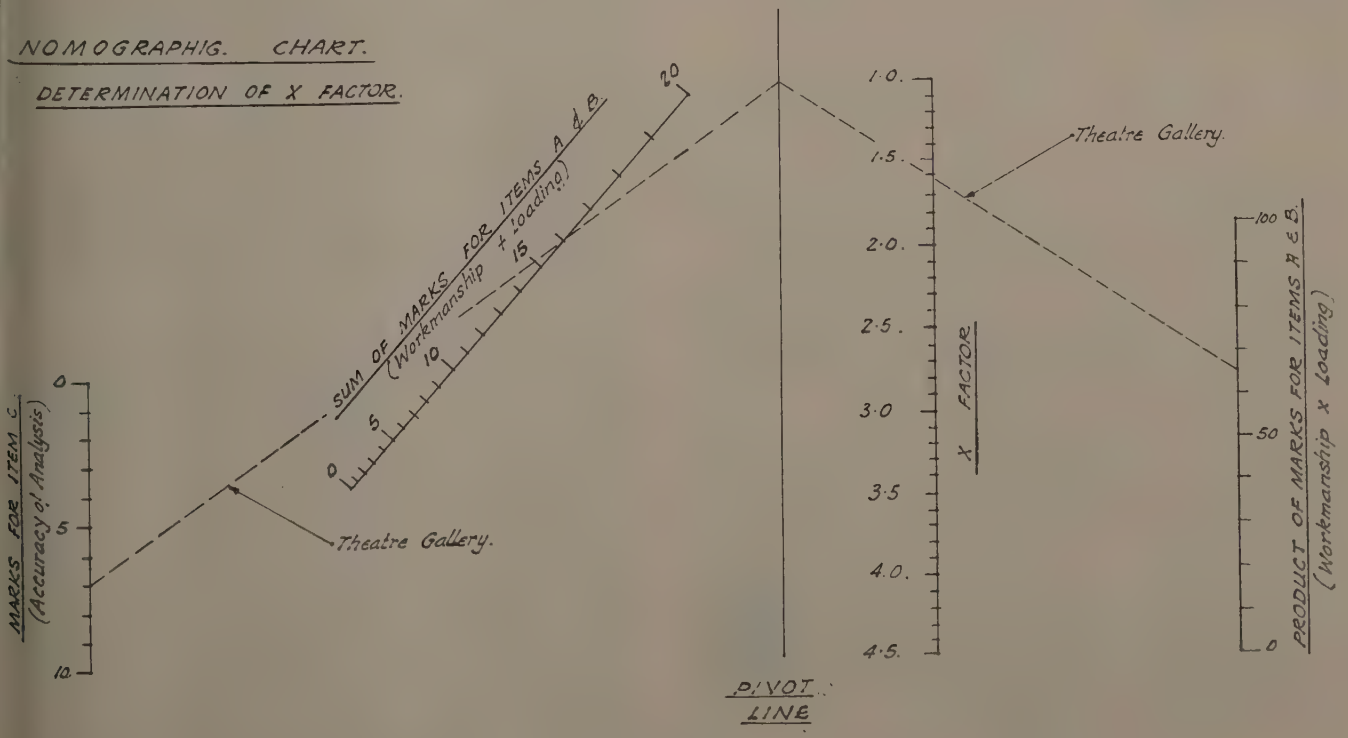

NOMOGRAPHIC CHART.
DETERMINATION OF X FACTOR.
MARKS FOR ITEM C (Accuracy of Analysis)
SUM OF MARKS FOR ITEMS A & B (Workmanship + Loading)
Theatre Gallery.
X FACTOR
PIVOT LINE
PRODUCT OF MARKS FOR ITEMS A & B (Workmanship x Loading)

Chart 1.

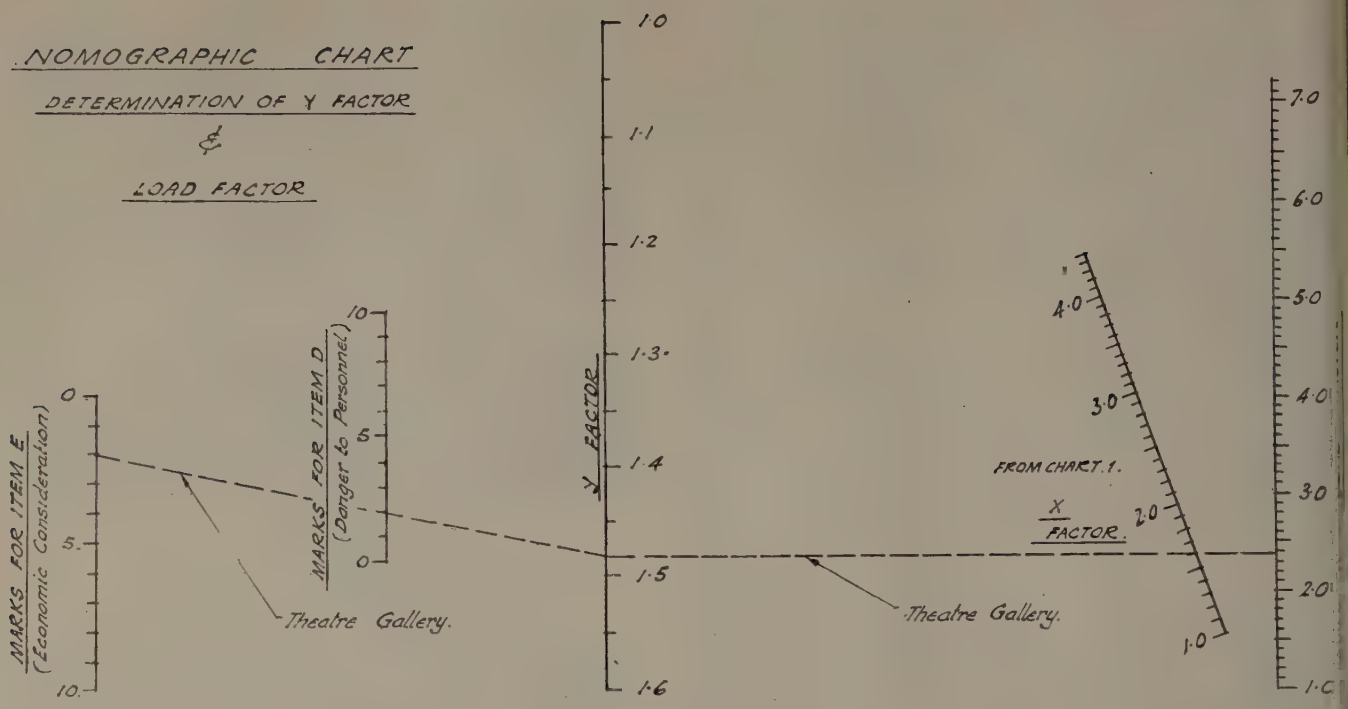


Chart 2.

In Chart 2, a straight-edge placed on the above values of the first two scales gives a Y factor of 1.48. If the straight-edge is pivoted around this point on the Y scale until it passes through 1.61 on the X scale, it will indicate an ultimate load factor of 2.38 on the end scale.

3. *Notes on use of method*

The nomograms are more sensitive to variations in the five characteristics than is the Tabular System, but such sensitivity may not be necessary. Though simple in operation, it is not as simple as the Tabular System, and the basis of its construction is not so rapidly apparent.

Appendix IV

The direct product system, or the evaluation of ultimate load factors by the product of factors for separated influences

This system has much in common with the Tabular System of the report but attempts to make (1) separate instead of collective assessments for Items A, B and C of Group X, (2) separate instead of collective assessments for such unrelated components as materials, fabrication, construction, inspection and maintenance which are embodied in Item A and (3) separate instead of collective assessments for items in Group Y. There is also a slight difference in the treatment of the Y factors.

The X Group can be treated as covering as many parameters or influences as may be considered necessary. For the purpose of this Appendix the following six are taken :—

$A1$. Reliability of materials and their inspection.

$A2$. Standard of workmanship in shop fabrication.

$A3$. Standard of workmanship in site construction.

$A4$. Standard of maintenance, having regard to possibilities of deterioration.

B. Accuracy of design loading.

C. Accuracy of design.

X Factor

In the ' perfect ' structure :—

(*a*) The factors assigned for each influence wou be 1.0, and

(*b*) The overall X factor would therefore also be 1

(*c*) The sectional areas (or other governing pr perties) of the members of the structure wou be the minimum necessary to prevent collaps

Since such perfection is impossible, the questio to consider when assessing the factor for each of t six parameters is :—

" By what factor should the minimum areas (other properties) be multiplied in order to correct f deficiency from perfection ?"

The product of these six factors would give a loa factor making good all six possible deficiencies a becomes the X factor. It is a question whether this factor should be reduced on account of the unlikelihoo of all deficiencies arising together. There is this be noted, however, that with a reduced X fact premature failure *could* occur, whereas with t unreduced factor it *should not* occur. The unreduc factor is adopted here.

The condition of the ' imperfect structure ' wi an X load factor applied in this way should then similar to that of the ' perfect structure ' with a loa factor of 1.0. In other words under normal workin load the structure designed to the X—factored loa *could be* just on the point of failure.

It should be noted that the factors in the X Grou are not necessarily always greater than 1.0. It conceivable, for example, in the case of certain r dundant structures where exact analysis may n be possible or practicable that a method may applied knowingly to give conservative results. such cases the factor for ' accuracy of design ' mig conceivably be less than 1.0 and certainly should n exceed it.

There is no doubt that a high standard of engineerin judgment is called for in the above assessment proces

t is particularly important, to avoid undue multiplication of errors by the product system, to ensure hat in assessing each of the six factors no *systematic* rrors are introduced as, for example, by a tendency o assess each factor on the safe side.

Y Factor

The *Y* factor is intended to provide a margin of trength or safety to the ' perfect structure ' with an *X* factor of 1.0, or to the equivalent ' imperfect *X*—actored structure.' Whatever the margin it should e sufficient to relieve the designer of all fear of failure. To that extent it would seem that a case could be made or a constant *Y* factor; nevertheless it is in the nature of the human mind to require a margin of safety measuring up to the seriousness of the consequences of failure.

The consequence seeming to demand the greatest margin of safety is danger to human life, and if life is safe-guarded there appears no need to be concerned with lesser coincident consequences such as material loss, disruption of services, cost of replacement etc. However, where danger to life is *not* involved lesser consequences may in some cases be of sufficient seriousness to demand a margin of safety just as high.

For the purpose of illustrating the complete application of the direct product system, the *Y* factor is taken provisionally as lying between 1.15 and 1.35, with the provisos that the higher value is used wherever danger to life would be incidental to failure, and the lower value only where adequate warning of danger of failure can be ensured.

The results of applying the above method to the three examples dealt with in the Report are as follows :

	(a) R.C. Water Tank	(b) Steel Theatre Gallery Girder	(c) Brick Walls of large block of Flats
*A*1.	1.15	1.05	1.5
*A*2.	—	1.03	—
*A*3.	1.20	1.05	1.10
*A*4.	—	—	1.10
B.	1.0	1.15	1.10
C.	1.15	1.10	1.20
X Factor	1.59	1.44	2.38
Y Factor	1.20	1.35	1.35
Load Factor	1.91	1.94	3.21

Note :—The blanks in the above examples indicate that the respective parameters are considered as not applying.

Appendix V
Bibliography

Attention is drawn to the following publications relating to the problems discussed in the body of this report, but not necessarily followed therein. These publications themselves give numerous references to more detailed papers.

1 Tye, W. " Factors of Safety or of Habit." *Journal of Royal Aeronautical Society*, 1944.

2 Pugsley, A. G. " Concepts of Safety in Structural Engineering." *Proc. of The Institution of Civil Engineers*, 1951.

3 Johnson, A. I. " Strength, Safety and Economical Dimensions of Structures." *Royal Institute of Technology, Stockholm*, 1953.

4 Freudenthal, A. M. " Safety and the Probability of Structural Failure." *Proc. American Society of Civil Engineers*, Vol. 80 (Separate No. 468) 1954.

4 Prot, M. and Leor, R. " Conceptions Modernes Relatives á la Securité des Constructions." *Revue Generale des Chemins de Fer*, June, 1951.

5 Balaca, A. P. and Torroja, E. " Determinacion de Coefficients de Seguridad en las Distintas Obras." *Instituto Tecnico de la Construccion y del Cemento*, Madrid, 1950.

6 Torroja, E. and Paez, A. " Calcul de coefficient de Securité." *I.A.B.S.E. 4th Congress*, (1952) *Preliminary Publication* p.165.

Book Reviews

Appendix to the 11th Edition of How to Estimate for Building Work, by J. T. Rea. Compiled by A. E. Baylis. (London : B. T. Batsford, 1953.) 10 pp. and tables, 8½ in. × 5½ in. 3/6d.

This appendix to the current 11th Edition of Mr. Rea's book has been issued in order that the book may continue to be useful until a completely revised and rewritten edition is published in about two year's time. It gives details of increases in costs of materials and labour for the years 1936/7, 1939 and 1944 to July, 1953, inclusive, together with notes on the controls and regulations affecting civil building in force at the latter date.

Quality Concrete. The Concrete Association of India. (Bombay : The Association, 1951.) 217 pp ; 10 in. × 7½ in. Rs. 7/8 net.

This book attempts to cover a very wide field from the manufacture, handling, distribution of Portland Cement to its use in the manufacture of concrete and including the production and handling of aggregates, the design of concrete mixes, the testing of concrete and the effect of various substances on concrete with the relative recommended protective treatments.

Whilst much of the material included is available elsewhere, nowhere has an attempt been previously made to bring so much together. Students of building or civil engineering can find some excellent information in this volume and one feels that it is high time a similar type of book was made available for Building Students in this country.

One criticism which one might offer is that in any revision, an attempt should be made to give British practice in the methods of proportioning and designing concrete mixes and on questions of site control. The present volume deals mainly with the practice in the U.S.A. although the Specifications and Standards quoted are of The British Standards Institution.

W. H. R.

(*continued on page 165*)

The Flexibility Characteristics of Sheet Pile Wals

By P. W. Rowe, Ph.D., A.M.I.C.E.

Summary

RECENT research[1] has shown that flexibility is desirable for sheet pile wall sections. The flexural properties of steel, reinforced concrete, and prestressed concrete sections are studied, in order to determine the most flexible section for a given moment of resistance. General relationships between unit moment of resistance and the flexibility number of the piling are developed for the conditions of working stress, yield, and ultimate failure.

1. Introduction

The anchored sheet pile wall, shown in Fig. 1(a) is subject to a bending moment distribution, Fig. 1(b), the shape of which depends on the stiffness of the subsoil and the flexibility of the pile section. For

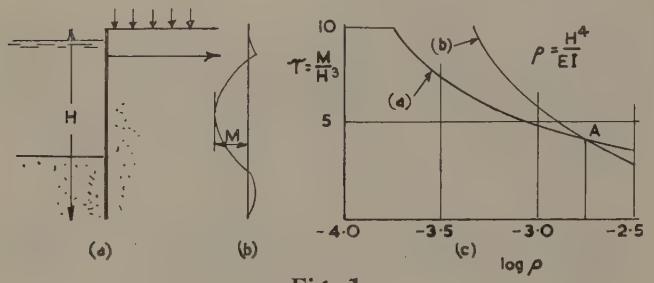


Fig. 1

a given subsoil, the maximum bending moment M decreases with increase in pile flexibility after the manner of curve (a) Fig. 1(c) to about 30 per cent of the value which acts on a stiff wall. Curve (a) is called the Operating Curve for the structure and is independent of the scale of the structure, provided the moment is plotted in terms of the unit moment $\frac{M}{H^3}$ or τ, and the section modulus in terms of the flexibility number $\frac{H^4}{EI}$, or ρ.

The designer requires to know the relation between the unit moment a section can withstand and its flexibility number. For wooden rectangular sections this relation is readily established.

Taking a section d inches deep, 1 foot long

$$I = d^3 \text{ in.}^4 \text{ per ft.}$$

$$M = \frac{I}{y} f = 2fd^2 = 2fI^{2/3} \text{ lb. in. per ft.}$$

Writing $\rho = \frac{H^4}{EI}$ and $\tau = \frac{M}{H^3}$ using the practical units of H, ft., E, lb./in.^{2}, I, in.^{4}/ft., M, lb.in./ft.

$$\tau = \frac{2f}{E^{2/3}} \times \frac{1}{\sqrt[3]{H\rho^2}} = \frac{\psi}{\sqrt[3]{H\rho^2}} \quad . \quad . \quad (1)$$

The quantity ψ is called the Flexibility Characteri ic of the section, and when the value of this is determi d for a given section the relation according to equa n (1) may be plotted as for example, curve (b), Fig. 1 . This is called the Structural Curve since it states unit moment a section of given flexibility can w - stand and refers to the behaviour of the structu e. The position of these curves along the abscissa va es as the cube root of the scale.

The point of intersection of the two curves and (b), Fig. 1(c), where the operating and resist g moments are equal, determines the design secti . The further a structural curve lies to the right on diagram, the smaller is the design bending moment, that the designer wishes to consider the wall sect n which gives the highest value of the characteristic in equation (1).

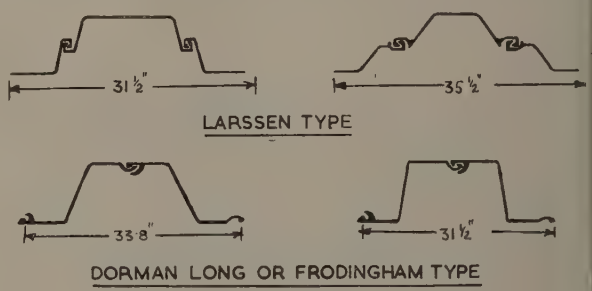


Fig. 2

The value of the characteristic increases with crease in working stress and decrease in Youn s Modulus of the wall material. It also depends on t e shape of the section and the position of the neut l axis. It is proposed to investigate the possible ran e of ψ values for steel, aluminium alloy, reinforc concrete and prestressed concrete sections.

2. Steel Sections

Examples of available sections are shown in Fig. These shapes have been obtained from both practi considerations in driving, and from the theoreti requirement of a high modulus/weight ratio, assumi a constant design bending moment. While it is n seen that the moment is not constant independent the section, Fig. 1(c) shows that the rate of mome decrease with flexibility for the operating curve is l than that for the structural curve. For this reason is still preferable to choose a section with a hi modulus. However, the designer requires to kn the structural curve for each type of section, and view of the variety of available section shapes it is interest to reconsider the theoretical aspects of t optimum section shape.

Consider a simplified section Fig. 3, of thickne αD, equal in weight to a flat plate of the same leng but of thickness D. The ratio L/D determines t frequency of the corrugations independent of scale, a is called the Aspect Ratio γ. Equating the areas

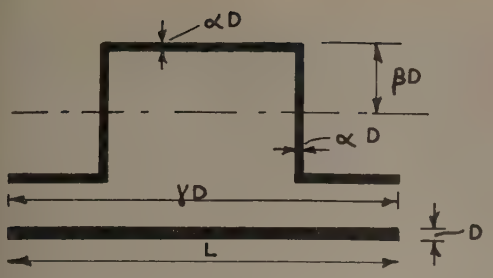


Fig. 3

he flat plate and corrugated section we have

$$\gamma = 2\alpha\frac{(2\beta - \alpha)}{(1 - \alpha)}$$

he second moment of area is found to be

$$I_s = k \times \frac{1}{12}LD^3 = kD^3 \text{ per ft.} \quad . \quad . \quad (2)$$

where

$K =$

$$\alpha\left[3(2\beta-\alpha)^2+\alpha^2+\frac{1}{\gamma}\left\{16(\beta-\alpha)^3+6\alpha(2\beta-\alpha)^2+2\alpha^2\right\}\right]$$

The coefficient k is called the Shape Factor, since it depends on both the frequency and the amplitude of the corrugations.

Following the derivation of equation (1) we obtain

$$\psi = \frac{f}{\beta}\sqrt[3]{\frac{k}{E^2}} \quad .$$

Choosing a series of values of γ and β, the values of α, k and ψ may be calculated and the structural curves drawn as shown in Fig. 4. These curves represent the locus of sections of equal shape but of varying weight. It is also of interest to plot the locus of sections of equal weight and varying shape.

We have from equation (2) that

$$\rho = \frac{\rho f}{k} \quad .$$

Starting with any size of flat plate of flexibility number ρf, thus fixing the weight, the chosen value of ρf is divided by the particular value of k for each structural curve and the point of intersection of this ρ value on the respective structural curve is plotted. The line through these points is the required locus for equal weight and varying shape. It is seen in Fig. 4 that such a locus line may be nearly parallel to the operating curve, so that the value of a high section modulus at the expense of flexibility is considerably less than under conditions of constant moment. In addition the more corrugated the section, the greater is the surface area exposed to corrosion and the smaller the metal thickness for a given weight. However, the factor of safety against ultimate failure is probably greater for the high modulus section since, as corrosion proceeds and the section deflects further, a larger decrease in operating moment may occur than for the flatter section. Advantage may be taken of this by designing to the structural curve at yield and increasing the wall thickness for a given rate of corrosion. It is clear, however, that the exact section shape is not a critical factor in the design.

A limit to the modulus/weight ratio is set by the buckling of thin sections, the requirement of an interlock, together with practical considerations of manufacture and driving. It is useful therefore to consider the characteristics of the types of section at present available. The different section moduli are generally obtained at approximately a constant value of $L = 31$-36 inches, as in Fig. 2, so that the aspect ratio must decrease with increase in section weight. Therefore the mean characteristic of all sections of a particular make cannot be strictly on a curve of the form given by equation (1). In order to study the optimum shape of section available it is preferable therefore to imagine that each section may be scaled up or down at constant shape as indicated in Fig. 5(a).

Rewriting equation (1) in terms of the given values of modulus z and fibre distance y for the section

$$\psi = \tau\sqrt[3]{H\rho^2} = \frac{M}{H^3}\cdot\sqrt[3]{\frac{H^9}{(EI)^2}} = \frac{f}{E^{2/3}}\times\left[\frac{z}{y^2}\right]^{1/3} \quad . \quad (3)$$

For $f = 8$ Tons/in.^{2}, $E = 30 \times 10^6$ lb./in.^{2}

$$\psi = 0.185\left[\frac{z}{y^2}\right]^{1/3} \quad . \quad . \quad . \quad . \quad . \quad (4)$$

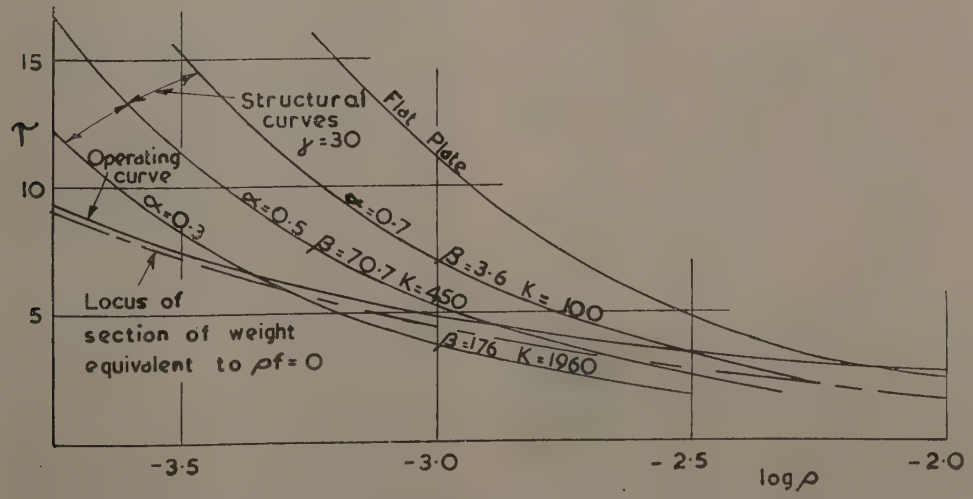


Fig. 4

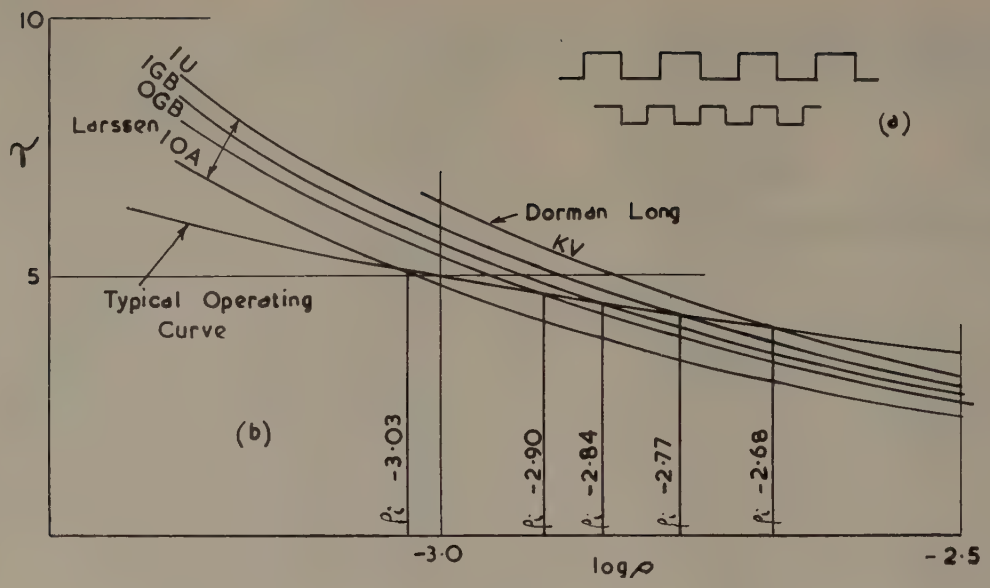


Fig. 5

For an average value $H = 40$ ft.,

$$\tau\rho^{2/3}=0.292\psi \qquad . \quad . \quad . \quad . \quad . \quad (5)$$

The structural curves for certain sections are plotted from equations (4) and (5) in Fig. 5(b). On the diagram an average operating curve has been drawn for use in comparison of the sections. The design flexibility for each is given at the intersection point of the curves at flexibility ρi, and this value is tabulated in Table I for each section. The sections available will give flexibility numbers ρ at finite intervals and it will not be possible to choose one such that $\rho=\rho i$ exactly. For cost comparison, the cost of the section after scaling at constant shape from flexibility ρ to ρi must be determined. The cost depends upon the thickness D of the equivalent flat plate.

Weight per sq. ft. $W=490\times 1^2\times\frac{D''}{12}=40.8\ D$ lb./ft.^{2}

$$W=40.8\sqrt[3]{\text{I plate}}=40.8\sqrt[3]{\frac{\text{I section}}{k}}$$

$$=40.8\sqrt[3]{\frac{H^4}{Ek\rho}}\ .$$

Therefore, for constant value of H and k

$$W\ \alpha\ \frac{I}{\rho^{1/3}}$$

so that the design weight W at flexibility ρi is related to the chosen section weight W_S at flexibility ρ by

$$W=W_S\sqrt[3]{\frac{\rho}{\rho i}}$$

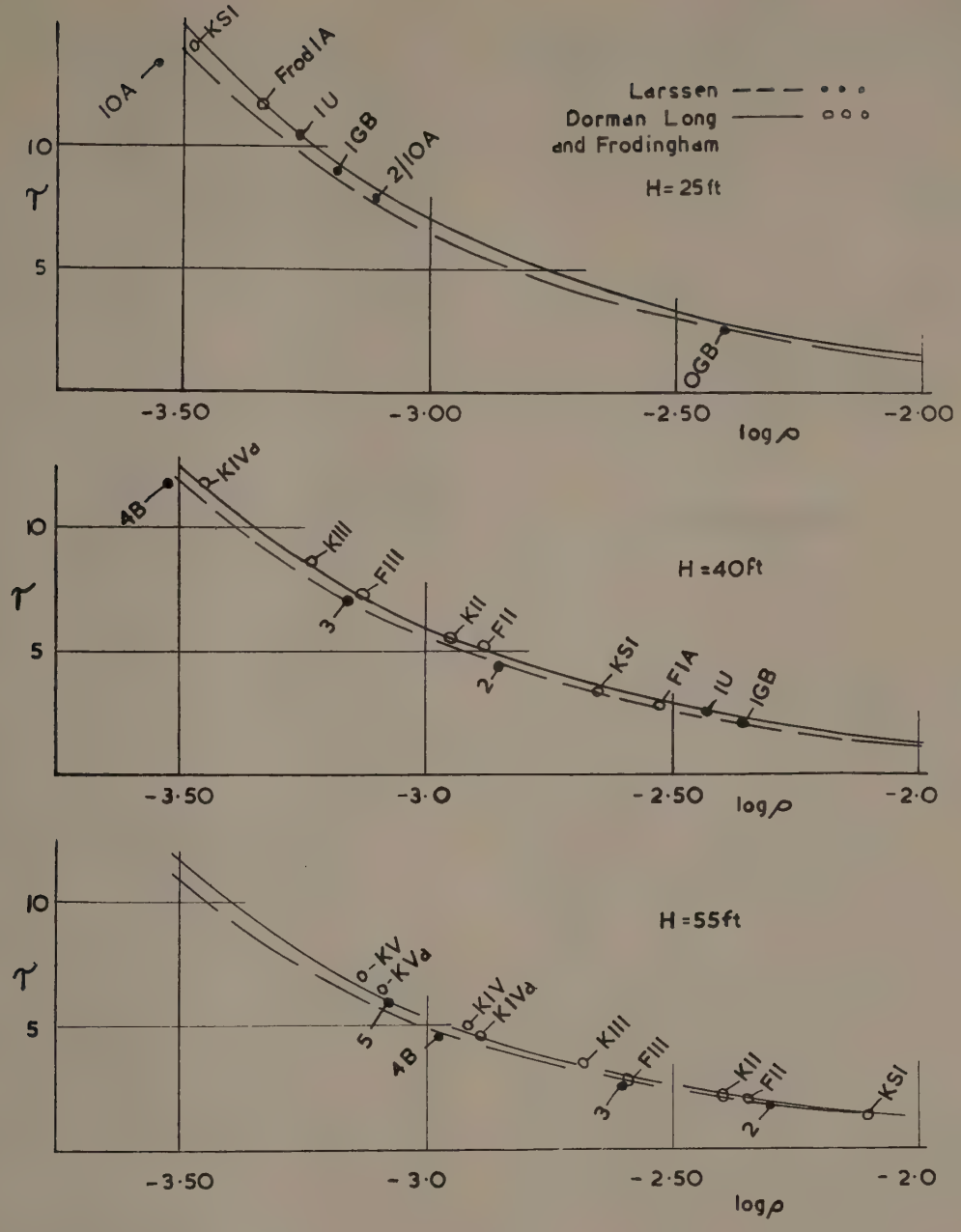
Values of log ρ, W_S and W are included in Table I.

TABLE I

Properties of Existing Sections

	Section	log ρ	weight *Ws*	ψ	log ρ*i*	*W*
Larssen	0 G B	—1.578	11.47	0.186	—2.90	31.7
	1 G B	—2.370	18.50	0.196	—2.84	27.4
	1 U	—2.436	21.70	0.206	—2.77	28.0
	2	—2.864	24.98	0.186	—2.90	25.7
	3	—3.160	31.74	0.189	—2.90	26.0
	4 B	—3.526	40.90	0.180	—2.93	25.6
	5	—3.640	48.74	0.197	—2.83	26.2
	10 A	—2.726	27.30	0.168	—3.03	34.4
	2/10A	—2.279	26.30	0.196	—2.84	40.5
Dorman Long	K S I	—2.657	21.71	0.199	—2.83	24.8
	K II	—2.960	24.78	0.200	—2.81	22.0
	K III	—3.238	32.56	0.207	—2.77	22.7
	K IVa	—3.446	38.09	0.208	—2.77	22.5
	K IV	—3.480	41.17	0.214	—2.73	23.0
	K V a	—3.529	48.54	0.213	—2.73	26.2
	K V	—3.692	54.27	0.220	—2.68	25.0
Frodingham	I A	—2.532	18.25	0.198	2.83	22.9
	II	—2.898	24.17	0.207	—2.77	21.8
	III	—3.148	31.50	0.202	—2.81	23.3

The optimum shape is given by the lowest value [illegible] *W*. This is not quite the same as the highest val[illegible] of ψ due to variations in the percentage weight of met[illegible] in the interlock and variation in thickness with[illegible] each section. The known fact that sections with t[illegible] interlock at the extreme fibre show better modulu[illegible] weight ratios than for sections with the interlock [illegible] the neutral axis is reflected in the values of ψ and [illegible] Of the former, seven of the sections show values of [illegible] between 22 and 23. This consistency occurs in spi[illegible] of a variable aspect ratio due to variation in t[illegible] coefficients β and α, but this is not achieved in all t[illegible] sections. It would seem theoretically preferable [illegible] standardize a few of the optimum shapes and provi[illegible] a range of sections at various scales. This wou[illegible] occasionally save 16 per cent weight in steel if theret[illegible] a section, $W=22$ lb./ft.^{2} were achieved instead [illegible] $W=26$ lb./ft.^{2}.



Figs. 6

Table I shows average values of ψ for Larssen and Dorman Long type piling respectively as 0.205 and 0.190. The corresponding τ, ρ relations are shown in Figs. 6 for three possible structure heights, together with the actual position of each section. It is evident that all the sections lie close to the mean curves for a constant shape factor, so that for the practical purpose of selecting a section, one structural curve for steel piling of any given height may be drawn taking $\psi=0.20$, and using equation (1).

Drawing Office procedure would be simplified if handbooks on sheet pile sections would include a list of flexibility numbers for a possible series of wall heights. The structural curves at a series of H values may be drawn on tracing paper for general use and the operating curve for each design orientated beneath. The section with the nearest flexibility number to the design section may then be chosen from the handbook.

It is evident that further moment reduction is obtained by using higher strength steel. Especially is this valuable with weak subsoils where the operating curve is displaced to the right on the diagram such as Fig. 1(c). Also the structural curves for steel at yield are necessary to determine the actual factor of safety against yield. Taking the yield stress as 15 tons/sq. in. for steel, specification 28/33 tons/sq.in. ultimate, and 23 tons/sq.in. for specification 37/43 tons/sq.in. ultimate, the ψ values are 0.375 and 0.575 respectively and the curves are plotted in Fig. 9. A general comparison with other wall materials is made in section 6.

3. Aluminium Alloy

This material is usually only likely to prove competitive with steel when the weight of the structure is a major consideration. However, the low Young's Modulus results in increased flexibility and the structural curve for W.P. Alloy 15 tons/sq.in., 0.1 per cent proof stress, shown in Fig. 9, assuming the same shape factor as for steel piling, gives an improvement over even the higher strength steel at yield. In addition

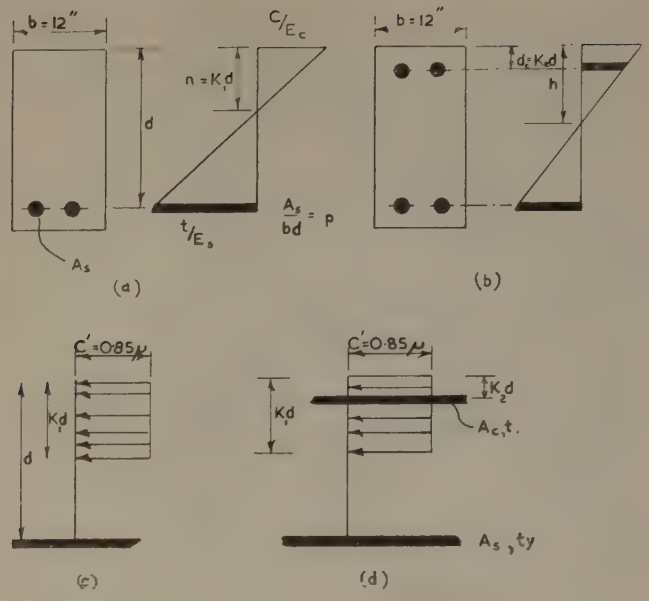


Fig. 7

the resistance to corrosion is greater than for steel. The cost ratio of aluminium to steel is nevertheless prohibitive at present, but should it fall, an aluminium sheet pile might well prove the best answer to medium scale walls driven into a soft silt subsoil.

4. Reinforced Concrete

The value of the characteristic ψ is calculated at working stress and at first yield conditions, using the straight-line, no-tension theory, and at ultimate failure using plastic theory. It has been usual to use equal tension and compression reinforcement to withstand a reversed type of bending moment distribution of unknown quantity and also to withstand tensile stresses during driving. However the bending moment distribution on the chosen section may now be calculated and the reinforcement more accurately detailed with secondary reinforcement for the driving stresses. Therefore the cases of the singly reinforced section and the section with compression reinforcement are analysed.

A 12-inch width of wall is considered, (see Fig. 7) all bending moments and second moments of area being per ft. run. The notation used is illustrated in Figs. 7. As in equation (2) we can write

$$I_C = kd^3 \text{ in.}^4/\text{ft.}$$
and $$M_R = \bar{k}d^2 \text{ lb.in./ft.}$$

Following equation (3)

$$\psi = \frac{M}{(E_C I_C)^{2/3}} = \frac{\bar{k}}{(E_C k)^{2/3}} \quad . \quad . \quad . \quad (6)$$

The values of $\bar{k}$ and k are now calculated for the possible section designs. The theory is well known so that the equations need only be stated.

(1) Working stress conditions

(a) Singly reinforced section Fig. 7(a)

$$\text{The steel ratio is } p = \frac{m}{2t_1(t_1+m)} \quad . \quad . \quad (7)$$

where t_1 = stress ratio t/c

$$\text{The moment modulus is } \bar{k} = 12pt\left(1-\frac{k_1}{3}\right) \quad . \quad (8)$$

where the neutral axis depth ratio is

$$k_1 = mp\left[\left(1+\frac{2}{mp}\right)^{\frac{1}{2}}-1\right] \quad . \quad . \quad . \quad (9)$$

$$\text{also } k = 4k_1^3+12mp(1-k_1)^2 \quad . \quad . \quad . \quad . \quad . \quad (10)$$

Hence for given design stresses or using a given steel ratio p, the structural curve is plotted from equations 8, 9, 10 and 6.

(b) Section with compression reinforcement Fig. 7(b)

Equating forces

$$\tfrac{1}{2}bnc + A(m-1)\,c\frac{(n-d_C)}{n} = At$$

$$\text{or } 2p\left[t_1-(m-1)\left(1-\frac{k_2}{k_1}\right)\right] = k_1 \quad . \quad . \quad . \quad (11)$$

Equating strains

$$k_1 = \frac{1}{1+t_1/m} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

Eliminating k_1 from equations (11) and (12)

$$\frac{1}{p} = \left(\frac{t_1}{m}\right)^2\left[m+k_2(m-1)\right] + \frac{t_1}{m}\left[2k_2(m-1)+1\right]-(m-1)(1-k_2) \quad . \quad (13)$$

Taking moments

$$\bar{k} = 12pt\left[(1-k_1)+\frac{(m-1)}{m}(k_1-k_2)\right] + \frac{4k_1^3 t}{m(k_1-k_2)} \quad . \quad . \quad . \quad . \quad . \quad (14)$$

and

$$k = 4k_1^3+6p\left[(m-1)(k_1-k_2)^2+m(1-k_1)^2\right] \quad . \quad (15)$$

The value of E_c assumed in the design does not have as critical an influence on ψ as would appear from equation 6, since the modular ratio m occurs in the expressions for $\bar{k}$ and k. For example, taking $p=0.018$ and $k_2=\frac{1}{10}$ for a section with compression reinforcement $\psi = 8\times10^{-6}t$ for $E_c = 5\times10^6$ lb./in.^{2}, and $\psi = 9\times10^{-6}t$ for $E_c = 2\times10^6$ lb./in.^{2}.

The equations (7) to (15) may also be used at steel yield provided the concrete stress has not exceeded one half the crushing strength. This will generally apply to factory produced piles. Alternatively, a limiting concrete proof stress may be chosen as the yield point for over reinforced beams.

2. Ultimate Stress Condition

Using Whitney's theory, the average concrete stress at failure C' is related to the prism strength μ' by

$$C' = 0.85\mu'$$

The prism strength is related to the cube strength μ by

$$\mu' \simeq 0.85\mu$$

Thus

$$C' \simeq 0.72\mu$$

The cube is normally tested at 28 days from casting. Assuming that the wall is not completed until 3 months at least from the date of casting, the actual strength will be approximately 20 per cent greater, so that

$$C' = 1.2 \times 0.72\mu = 0.87\mu$$

It is proposed to use the value

$$C' = 0.85\mu$$

In order that the formulae will have their familiar appearance, with the exception that μ refers to the 28 day cube strength

(a) Singly reinforced section Fig. 7(c)

For under-reinforced beams

$$M = pt\bar{y}\, bd^2(1-0.5k_1) \text{ or } \bar{k} = 12pty\left[1-0.5k_1\right] \quad (16)$$

where $k_1 = \dfrac{pty}{0.85\mu}$ . . . . . (17)

and $k = 4k_1^3 + 12m'p(1-k_1)^2$

The first term is small in this equation so that we may write

$$k \simeq 12m'p(1-k_1)^2 \quad . \quad . \quad . \quad . \quad . \quad (18)$$

where m' = ratio of the chord moduli at failure. According to Baker[2]

$$E_c' = 500C' \quad . \quad . \quad . \quad . \quad (19)$$

The ratio of the strains at failure is $\dfrac{\varepsilon t}{\varepsilon c} = \dfrac{1-k_1}{k_1}$

so that $\dfrac{ty}{E_s'} \times \dfrac{E_c'}{C'} = \dfrac{ty}{m'C'} = \dfrac{1-k_1}{k_1}$ . . (20)

Substituting (16), (18), (19) and (20) into equation (6) we obtain

$$\psi = \frac{(1-0.5k_1)}{27.5}\left[\frac{p\ ty}{k_1^2(1-k_1)^2}\right]^{1/3} \quad . \quad (21)$$

(b) Section with compression reinforcement Fig. 7(d)

The compression steel is very close to the neutral axis at failure and just commences to yield at ultimate collapse. Let the stress in the compression steel just prior to collapse equal t.

Equating strains

$$\frac{C'}{E_c'k_1} = \frac{t}{E_s(k_1-k_2)} \quad . \quad . \quad . \quad . \quad (22)$$

Equating forces

$$A_s ty = A_c t + C'\left[k_1\, bd - A_c\right] \quad . \quad . \quad (23)$$

Writing $A_s = A_c = pbd$, substituting for t from equation (22) and E_c' equation (19), and inserting $E_s = 30 \times 10^6$ lb./in.^{2} we have

$$k_1^2 + k_1 \frac{2p}{1.7\mu}\left[60{,}000-(ty+0.85\mu)\right] - \frac{2pk_2}{\mu} \times 35300 = 0 \quad . \quad . \quad . \quad . \quad (24)$$

Taking moments about the neutral axis

$$\bar{k} = 12p\,ty\left[(1-k_1) + \frac{6\times10^4}{ty}\frac{(k_1-k_2)^2}{k_1} + \frac{0.85\mu k_1(k_1-p)}{2p\ ty}\right] . \quad . \quad . \quad . \quad . \quad (25)$$

Following equations (15) and (18)

$$k = 12m'p\left[(k_1-k_2)^2+(1-k_1)^2\right] \quad . \quad . \quad (26)$$

Substituting equations (19), (25) and (26) into equation (6), neglecting the term (k_1-k_2) which is very small at failure and writing $(k_1-p) \simeq k_1$ we obtain

$$\psi = \frac{(1-k_1^2)}{27.5}\left[\frac{p\ ty}{k_1^2(1-k_1)^2}\right]^{1/3} \quad . \quad . \quad (27$$

A number of theoretical designs have been investigated using the above equations and three examples are summarised in Table II.

TABLE II

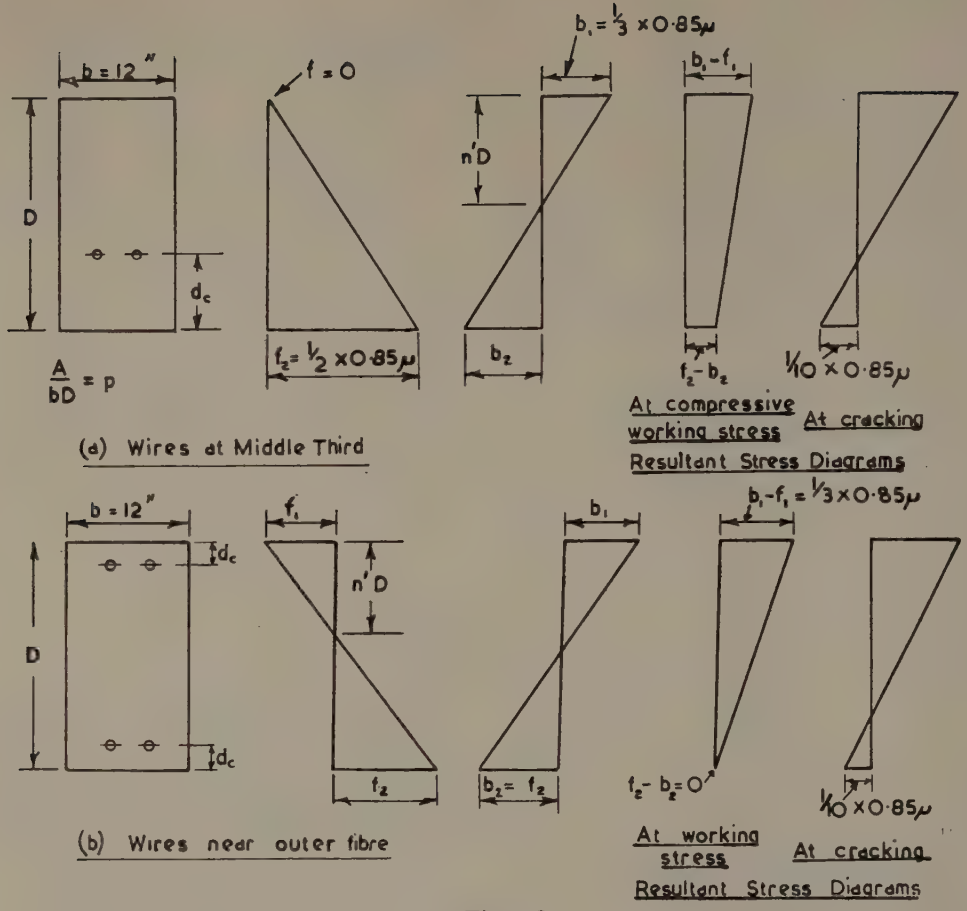
Reinforcement	Stress	μ lb./in.^{2}	p	t lb./in.^{2}	C lb./in.^{2}	ψ
Single M.S.	working	6,000	.018	18,000	1,250	0.18
	yield			37,000	2,600	0.38
	ultimate			37,000	5,100	1.26
Compression Steel M.S.	working	8,000	.036	18,000	1,190	0.25
	yield			37,000	2,440	0.52
	ultimate			37,000	6,800	1.39
Compression Steel Square Twisted	working	6,000	.018	27,000	1,380	0.25
	yield			60,000	3,070	0.55
	ultimate			60,000	5,100	1.17

The analysis shows

(1) that the optimum characteristic ψ is obtained using twist steel reinforcement in tension and compression ;

(2) that the characteristic is approximately the same at ultimate failure for most practical values of p and μ for either type of reinforcing.

Curves for sections with twist steel reinforcement are plotted in Fig. 9 together with examples of laboratory tests on models using mild steel reinforcement.



Figs. 8

5. Prestressed Concrete

The pretension system is likely to be the most economical for sheet pile production. Two systems of reinforcement are shown in Figs. 8, namely single wires at the one-third point, and wires at the outer fibre with tension reinforcement during prestressing. The usual equations required are

$$f_2 = 2p\,tp\,\frac{(1-n'p)}{(1-2n'p)} \quad . \quad . \quad . \quad . \quad . \quad (28)$$

$$\text{where } np' = \frac{(1-3dc/D)}{3\,(1-2dc/D)} \quad . \quad . \quad . \quad . \quad . \quad (29)$$

$$\text{and } M_R = \frac{bD^2b_1}{6n'}\left[\,3\,\frac{dc}{D}\,(1-2n')-1+3n'\right] \quad . \quad (30)$$

$$\text{where } n' = \frac{0.5+mp\left(1-\frac{dc}{D}\right)}{1+mp} \quad . \quad . \quad . \quad . \quad (31)$$

(a) Wires at the one-third point $\left(\frac{dc}{D} = \frac{1}{3}\right)$

(1) At working stress

From equ (30) $M_R = 2b_1\,D^2$

$$\therefore\ k = 2b_1$$

also

$$k = 4\left[n'^3+(1-n')^3\right]+12(m-1)p\left[1-n'-\frac{dc}{D}\right]^2$$

The value of k is very close to unity, and taki

$$b_1 = \frac{1}{3} \times 0.85\,\mu$$

$$\psi = \frac{0.53\mu}{E_c^{2/3}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3$$

The value of E_c will vary between 4×10^6 a 6×10^6 lb./in.^{2} for high strength concrete, a ran which lies in the general relation[3],

$$E_c = 60{,}000\sqrt{\mu}$$

$$\text{Hence } \psi = .035\left[\frac{\mu}{10^3}\right]^{2/3} \quad . \quad . \quad . \quad . \quad (3$$

(2) At cracking

Assuming $f_2 = \frac{1}{2} \times 0.85\mu$

and tensile strength $= \frac{1}{10} \times 0.85\mu$

then crack occurs when $b_2 = 0.6 \times 0.85\mu$.

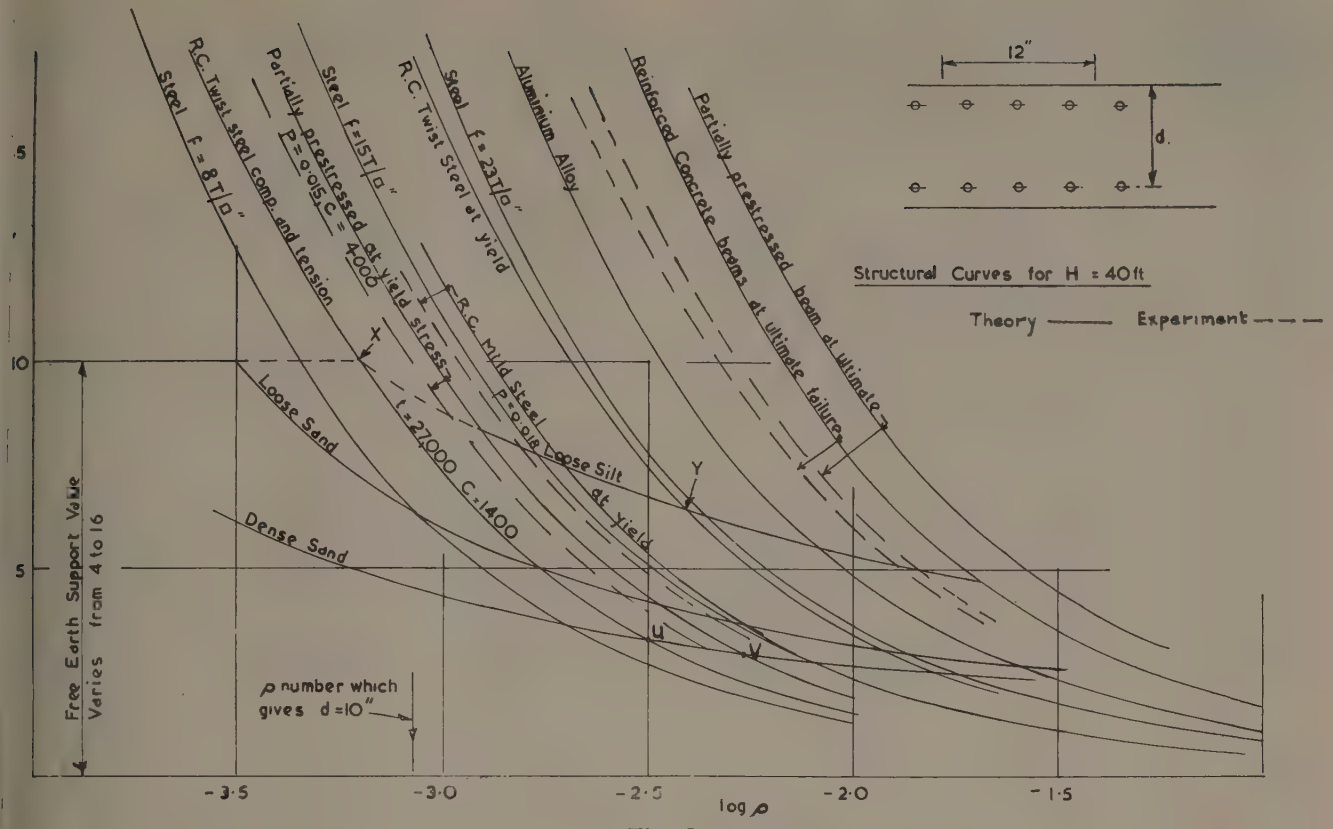


Fig. 9

Using $b_2=b_1\frac{(1-n')}{n'}$ and the following stresses ty=250,000 lb./in.², μ=7,000, lb./in.² we obtain p=.0084 and n'=0.518

whence $\bar{k} = 2.15b_2$

from equ (30)

and $$\psi = .072\left[\frac{\mu}{10^3}\right]^{2/3} \quad . \quad . \quad (34)$$

(b) Wires at the outer edge $\frac{dc}{D} = \frac{1}{10}$

(1) At working stress

The value of n' remains approximately equal to 0.52, so that putting $\frac{dc}{D} = \frac{1}{10}$ into equation (30)

$\bar{k} = 2.4b_2$

Also $k^{2/3} \simeq 1.1$

For $b_2 = \frac{1}{2} \times 0.85\mu$

$$\psi = .06\left[\frac{\mu}{10^3}\right]^{2/3} \quad . \quad . \quad . \quad . \quad . \quad . \quad (35)$$

(2) At cracking

$b_2 = 0.6 \times 0.85\mu$

$$\psi = .073\left[\frac{\mu}{10^3}\right]^{2/3} \quad . \quad . \quad . \quad . \quad . \quad . \quad (36)$$

The results are summarised in Table III.

TABLE III

dc/D		1/3		1/10	
Condition		at working stress	at cracking	at working stress	at cracking
μ	7,000	.128	.264	.219	.264
	10,000	.162	.334	.278	.334

Table III shows that the ψ values are higher with the steel at the outer edge. For this case the characteristic is similar to that for steel piling and less than that for reinforced concrete using twist steel. This is due to the higher value of E_c under prestressed working conditions. However, prior to cracking the corrosion rate for the prestressed section should be lower.

At ultimate failure the equations are very similar to those for reinforced concrete sections with an ultimate characteristic of the same order.

(c) Partial prestressing.

The stabilising influence of moment decrease after flexibility increase with cracking will lead to the use of a lower factor of safety against cracking. The use of partially stressed wires on the tension side will prevent the formation of large cracks and check further deterioration due to corrosion.

Fig. 9 shows some observed and theoretical structural curves. The model beams were 4 in. × 2 in. × 6 ft., subject to a reversed bending moment diagram similar to that to be expected to act on a flexible sheet pile wall. The cube strength of the concrete used was 12,000 lb./in.² at one week. The tests were made

under short period loading, and with time and increase in flexure, the curves in Fig. 9 would be expected to move to the right. The agreement between the models and the theory is sufficiently close to allow a general comparison of the flexibility characteristics of the types of wall.

6. Conclusions

The curves in Fig. 9 illustrate two new considerations. Firstly the real factor of safety against yield and ultimate failure should be considered using the appropriate structural curves for each particular design. For example, let us assume that the wall is 40 ft. high, that the conditions of penetration, surcharge etc. impose a maximum value of τ equal to 10 lb.in./ft.[4] and that the subsoil is a loose silt. The design flexibility for a R.C. section using twist steel at t=27,000 lb./sq.in. would be given by point $\times$. The nominal safety factor against yield would be about $^{60}/_{27}$. Now if an error is made in soil sampling or in the estimation of the loading, then the maximum τ value may exceed 10. If it exceeded 10 in the ratio $^{60}/_{27}$, yield would commence, so that in this respect the factor is the true value. However if the section were deliberately underdesigned such that the yield stress was reached, equilibrium would occur at point Y.

$$\text{Since } \frac{\text{operating moment at } Y}{\text{operating moment at } \times} = \frac{6.3}{10}$$

the overall factor against yield in the design for t=27,000 is $3\frac{1}{2}$. A definite outward wall yield will induce arching of the retained soil reducing the moment by 30 per cent. The factor against yield progressing is therefore $\frac{3.5}{0.7} = 5$. It follows that separate load factors should be applied to the calculation of the operating moment on the one hand and the allowable stresses on the other. This was first suggested by Brinch Hansen[4] who also uses separate factors for the soil constants and the estimated super loads.

Secondly, Fig. 9 suggests that the possibility of using concrete walls in place of steel is improved. Concrete piling has generally not been competitive with steel. This has been partly due to the use of relatively low concrete stresses with consequent heavy sections up to 18 in. thick, costly to handle and to drive. For the example in Fig. 9, the flexibility number is marked corresponding to an effective depth d=10 inches. The section size at any other flexibility varies as the cube root of the ratio of the flexibility numbers, so that for a dense sand subsoil at point

$$U,\ d=10\times\left[\ \frac{1}{\text{anti-log}\left(\frac{.57}{3}\right)}\ \right] = 6\tfrac{1}{2} \text{ inches.}$$

A prestressed design at point V gives $5\frac{1}{2}$ inches. Allowing for cover the overall thickness is 7 inches for the 40 ft. wall.

Acknowledgment

The work was carried out in the Engineering Laboratories Manchester University. The Author is indebted to Mr. H. Howe, B.Sc., for the model investigation of prestressed concrete sections.

Notation

A	denotes	area of tension or compression reinforcement
D	,,	depth of section
E	,,	Young's Modulus
H	,,	pile height
I	,,	second moment of area
L	,,	steel section length
M	,,	moment of resistance
W	,,	weight per square foot
b	,,	concrete section length
c	,,	concrete stress
d	,,	effective depth of section
d_c	,,	steel cover
f	,,	outer fibre stress
k	,,	the ratio I/D^3 or I/d^3
$\bar{k}$	,,	the ratio M/D^2 or M/d^2
k	,,	the ratio n/d
k	,,	the ratio dc/d
m	,,	modular ratio E_s/E_c
n	,,	depth to neutral axis
p	,,	steel ratio $\frac{A}{bd}$
t	,,	steel stress
tp	,,	steel prestress
ty	,,	steel yield stress
t_1	,,	stress ratio t/c
y	,,	depth to neutral axis
z	,,	section modulus
β	,,	the ratio y/D
γ	,,	the ratio L/D
μ	,,	cube strength
μ'	,,	prism strength
ρ	,,	flexibility number $\frac{H^4}{EI}$
τ	,,	unit moment M/H^3
ψ	,,	flexibility characteristic

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The Design and Construction of the Nuclear Reactor Buildings at Windscale Works, Sellafield*

Discussion on the Paper by Mr. D. R. R. Dick, B.Sc., M.I.C.E.

THE PRESIDENT, introducing Mr. Dick, said that he was the Superintending Structural Engineer who had been responsible for the design and the conduct of the very remarkable work of which he was going to speak. He was known well to many who were present that evening, so that he needed little introduction; in any case he was well able to speak for himself, and his work would speak for him too.

(Mr. Dick presented his paper.)

THE PRESIDENT, who welcomed the very pleasant duty of proposing a vote of thanks to Mr. Dick, said the meeting had listened to a lucidly delivered account of a truly remarkable piece of work.

Concrete, he continued, was a very useful material and it had many valuable properties. But it had also properties which were troublesome; it would shrink and creep, and on the slightest provocation it would crack. Yet Mr. Dick and his colleagues had had to produce great masses of concrete to fantastic tolerances; these must also be air-proof and water-proof when subjected not only to heat, but to steep thermal gradients. They must have felt themselves to be in a somewhat similar position to the skilful billiard sharp whom W. S. Gilbert's Mikado would condemn to play "extravagant matches

> In fitless finger stalls,
> On a cloth untrue
> With a twisted cue
> And elliptical billiard balls"

Yet they had won their match, and he was confident that the meeting would wish to pass with acclamation a very hearty vote of thanks to Mr. Dick for his informative explanation and clear illustrations of a remarkably interesting piece of work.

(The vote of thanks was accorded with enthusiasm.)

Discussion

MR. L. R. CREASY first offered greetings and an apology for absence from Mr. Greetham, the Chief Structural Engineer to the Ministry of Works. Perhaps, he said, others present would forgive him in saying that he and his colleagues looked upon the meeting as something of a Ministry occasion, and their Chief would certainly have been present had he not been unavoidably prevented from doing so.

It was also something of an historical occasion. It was perhaps in the middle of the last century when the structural engineer, with new roads and railways, first began to impinge so much on the community at large, but since that had also coincided with the birth of our engineering societies, our present-day very highly organised exchange of information was unknown and many details of those early structures went unrecorded. It looked rather as though, due to the political co-existence of today, a similar situation might develop in the present period which had introduced the structural engineer into the atomic field. Happily, however, some relief had been achieved and all were grateful for Mr. Dick's paper.

Even so, the author had chosen—in modesty, one suspected—to give a paper in very concise and almost frigid terms, and one would need to recall the background in order to gain a proper perspective of the circumstances in which the project was conceived and completed. One would need to throw one's mind back to those turbulent days at the close of the last war, when the Ministry of Works was instructed to build atomic reactors—knowing nothing of such matters, but with the doubtful encouragement, if such it be, that there were few others who could, from previous experience, contradict.

As an initial essay into these mysteries, a party, of which Mr. Dick was a member, was sent to Canada; and then, after some preliminary experience on the building of smaller research units at Harwell, Mr. Dick was outposted to Lancashire, as the Engineer-in-Charge responsible for the structural design of the very much more ambitious project at Windscale.

Some of the romance might be missing from the paper; perhaps only the onlooker could divine its presence; the political spectre hovering in the background with constant demands for a tight time schedule, the pivot position between the scientist in the back room and the construction team on site, resulting in a treadmill of constant striving to convert in so short a time the ideas of the former into designs of steel and concrete for the latter.

Perhaps it was inevitable that this first report should have been couched in such very matter-of-fact terms, and thus when readers had had the opportunity to absorb the information, Mr. Dick would, perhaps, prepare a sequel, setting out something of the heartbreaks, and even of the humour, provided by such a project. In the meantime, all were grateful for the paper; first as structural engineers and secondly, as members of a much wider community, that the finished product worked.

* *Read before the Institution of Structural Engineers at* 11, *Upper Belgrave Street, London, S.W.*1., *on the* 25*th November*, 1954. *Dr. S. B. Hamilton, M.Sc., B.Sc.(Eng.), A.R.C.S., M.I.C.E., (President) in the Chair. Published in The Structural Engineer, Vol. XXXII, No.* 11, *pp.* 287-303. *Nov.* 1954.

Mr. W. G. Carter, M.B.E., (Associate-Member) having had the privilege of working with Mr. Dick on the project, although in a relatively small capacity, said that as a result of that privilege he had some knowledge of the contents of the paper before he had read it, and his first reaction was that it was a masterpiece of understatement.

To Mr. Dick who had lived with them for seven or eight years, the problems, some only of which were set out in the paper, were a matter of everyday concern but to the average engineer they were on a level far above normal. It was something of a miracle that he had obtained satisfactory solutions. Tolerances were quoted of ± 0.015 ins. How was temperature movement catered for during construction?

Further, in this realm there was no precedent, no Code of Practice and Mr. Dick was working virtually in the unknown. Yet, as Mr. Creasy was able to say, the finished article was working. Again, Mr. Dick had throughout to work directly with and to conform to the requirements and dictates of the highest grade scientists. This, in itself, was by no means an easy task.

Finally, would Mr. Dick say why the main superstructure was of riveted construction—why not welded?

Mr. H. Kaylor (Associate-Member) said that in thanking Mr. Dick for his paper it must have been understood how much everyone appreciated that the Ministry had given them the privilege of learning something of the fine work that was described. They had read what was stated in the paper, and at the meeting they had learned something, from the quite simple explanation Mr. Dick had given, of how the reactor worked.

Putting some questions, Mr. Kaylor first asked, in view of the heavy loads that came down on to the foundation raft, which was of the order of 57,000 tons, whether cellular construction had been considered, because he believed the thickness of the raft was about 10 ft. After the start of the work some extra loading had to be carried through the chimney stack.

With regard to the construction of the roof to the reactor, he asked if it were ever considered practicable to use precast prestressed elements as permanent shuttering. Would that have been possible, in view of the pipes or tubes which were necessary and also in view of the suspension of the shield? Again, in regard to the same roof, in view of the difficulties experienced in pouring the concrete, he asked what consideration was given to the use of special concretes, such as Colcrete, because it was mentioned that finally pockets were filled by grouting. Further, was any special cement used, and could an expanding cement have been used with advantage?

Finally, if the question were not indiscreet, he asked whether some idea could be given of the actual temperature ranges that were involved so far as the concrete was concerned.

Mr. F. M. Bowen (Associate-Member of Council) said he hoped that if he bombarded the Lecturer with questions, Mr. Dick would take them as a mark of the great interest which the paper had for him and, doubtless, for everybody else present.

Concerning concrete, Mr. Bowen could not see an reference in the paper to the actual mixes used in t various parts and the minimum test strengths required could Mr. Dick give some information about tho matters and about the curing methods? He aske also if any particular test methods were used. seemed possible that something rather more rigid tha the normal test procedure may have had to b adopted in this case.

In regard to the portal frames and the arrangemen of the building, it had been said that the bottom 80 f of the inner legs of the central portal frames we built solidly into the Reactor concrete wall. H imagined from the slides that the wall would hav been amply strong to support the portal frames a the 80 ft. level and he asked if he was correct in assum ing that the reason why these legs were made the fu height was to provide shelter for and to enable th crane to be used in later work.

Referring to Figs. 5 and 6, Mr. Bowen said he wa intrigued by the two combined cases. Presumabl one of the most important factors was the differentia deflection and the consequent effect on the crane. H asked if any special arrangement was made in th bearings of the crane wheels to take up the latera deflection. He had noticed that in Fig. 6 the de flections for the combined cases 1, 3 and 4 were given If case 4 was omitted—and maybe case 3 could ac without case 4—the differential deflection would b worse. He asked if the combined cases illustrate were the only ones considered or had an intermediat grouping, allowing for vertical and crane loads acting at the same time as partial wind or full snow loac also been considered? Such an intermediate groupin was quite feasible in practice and probably woul give worse results than the cases shown.

Referring next to Fig. 7, he said he had observed the rather deep stools on which the purlins were fixed but from the drawings could not see any reason fo them except possibly to house the eaves cantilever and this alone did not appear to be an adequat reason for such stools.

He was rather shocked to see that in Fig. 8, 2 in of grout were specified under the portal frame bases In his opinion it was illogical and rather amazing that so often the weakest material (i.e. materia appreciably weaker than the concrete in the foundation itself) was placed immediately under the bases, where the concentration or intensity of loading was greatest For years his practice had been to use dry concrete packing and he would have thought that in this case in view of the very heavy loads involved and the close limits of permissible deflection, such dry packing would have been much more sound. Possibly i had in fact been used.

Finally, Mr. Bowen said that last year he had seen the building from a distance; fortunately the sun was shining and the structure was most impressive in appearance. He thought that Mr. Dick, his colleagues and all the Contractors concerned, not least those men who had built in the filters 300 ft. up had undoubtedly earned sincere congratulations on their fine achievement.

Mr. John King (Associate-Member) added his compliments to Mr. Dick on an excellent paper and put two questions:

In connection with the sub-soil investigation it was mentioned that 2½ to 3 tons per sq. ft. was a reasonable bearing capacity on that rather heterogeneous glacial material. Mr. King asked what was the method of testing used to determine this figure. The proving bores were taken to a depth of 40 ft. which seemed to him rather shallow, especially in a sand and gravel in which there were lenses of clay, and also when it is remembered that a pressure intensity of half the surface pressure can exist at depths greater than the 40 ft. mentioned from the loading conditions under this structure.

That thought led him to wonder, what were the reasons—no doubt they were good ones—whereby it was decided not to go right down to bed rock with some form of piling, caissons or other deep foundation, in view of the desirability to avoid differential settlement and having regard to the fine tolerance limits which were mentioned in the paper. It seemed to him that there might well have been a tendency to some differential settlement.

Mr. Dick, replying to the questions put to him so far, referred first to the President's question concerning thermal gradients and said he could not agree more with his comments. But on temperature stresses he did not know a great deal, and he would like to know somebody who did. The temperature gradients that were designed for and were experienced were not very high, they had made sure of that by means of insulation, cooling and the like. The gradients had varied throughout the job, from about 25° C. upwards to 35° or 40°C., and were largely determined by the fact that 1¼ in. and 1⅜ in. reinforcing bars used in the concrete were out of scale with the 9 ft. thicknesses of concrete required for shielding purposes.

The question of accuracy in the face of temperature variations was raised by Mr. Carter. Throughout the construction of the graphite mass and the channels in it they were working under a complete air conditioning system, which controlled the temperature. In the construction of civil works the tolerances were not so fine as for the graphite mass. Further, most of the really accurate work was done in very heavy concrete, which was not affected very much by day-by-day changes or even week-by-week changes; finally in levelling up off each concrete lift they were dealing only with a small height. There was obviously a factor there, but broadly speaking the accuracy was achieved, and they did take certain measures to allow for that kind of thing.

As to why the portal frames were not welded, he said they had had a welded design at one time; but having let the contract, and inasmuch as they were demanding a very high speed of construction, they had designed the members to suit the fabricator's facilities. There was quite a quantity of shop welding, although the actual flanges were riveted. They were not prepared to do site welding at a height of 140 ft.

Dealing with Mr. Kaylor's question concerning cellular construction of the raft, Mr. Dick said he had mentioned the very high shears in the 10 ft. thick raft, and if a cellular raft had been used the shears would have been quite prohibitive. On other work at Windscale, in the chemical group, where there was one particular building 200 ft. high and with a chimney going up to 400 ft., they had used cellular construction. That particular raft was carrying something like 27,000—28,000 tons. The one next door was also of cellular construction for a similar reason.

In connection with the references to precast prestressed beams, Colcrete and special cements, he said that the client, Sir Christopher Hinton, held very strong views on the lines that there was already enough experiment in connection with nuclear energy, without starting to experiment on the construction work. That might sound like dismissing such points rather light-heartedly, but there was quite a lot in it. Certainly for the roof over the reactor, with its tubes, it would have been difficult to work with any prestressed system.

Apart from reactors there were many other structures containing radioactive sources where prestressed concrete post-tensioned beams had been used, so that that particular approach had been used.

Coming to Mr. Kaylor's questions on mixes of concrete and the concrete strength, he said that one of the requirements in shielding against neutrons was that there must be light elements—hydrogen principally—chemically included in the concrete. Generally speaking, they had worked throughout to approximately a 1 : 2 : 4 mix.

Nothing very special was done about curing. Shrinkage was allowed for on the basis of keeping the limits of the bays down to about 25 ft. A great deal of more sensitive work was done inside the cladded buildings.

With regard to tests on concrete, he would say this about cube tests. They indicated that one was getting some good concrete from the mixer; but did it follow necessarily that one was getting a good concrete in place? Other than using radioactive sources—which they had used—they had not found any satisfactory means of testing apart from close supervision of the work during construction.

The portals were concreted in ultimately; but the important requirement was to enclose the work within the buildings and to provide overhead coverage during the operations.

With regard to the deflection of the portals, particularly at crane level, it was specified that ¾ in. was the maximum gauge widening, but when the crane was supplied it was found that it would have been possible to tolerate much greater deflection at crane level.

He could not go into questions of differential deflection from the diagrams; as not all the variants considered had been included in the Paper. The stools under the purlins were to accommodate starred angle bracing.

On the question of 2 in. of grout, he said the loads under the stanchions were not very great; principally it was the moments that were involved. Grouting had in fact been carried out with concrete using a small size of aggregate.

Dealing with Mr. King's question of bearing tests on the ground he said that the settlement tests were necessarily based on an assumed thickness of clay lenses underneath. The actual pressures adopted were based partly on these tests and partly on engineering experience.

On the question of proving bores beyond 40 ft., he said bores were carried down to 100 ft. to prove the general characteristics of the sub-strata. Bores at close centres down to 40 ft. were sunk because it was felt that clay lenses were likely to be serious only in the depth where there were fairly high pressures. In point of fact the settlement had been about $\frac{5}{8}$ ins. and was uniform under each raft.

Those remarks also answered the question as to whether piles or anything else could have been used. There was no point in going down further if one did not have to do so.

Mr. J. A. G. Smith (Member), adding his congratulations to Mr. Dick on having carried through the work described to a very satisfactory conclusion, said he was particularly anxious to do so because the firm with whom he was employed were at present putting up a boiler plant at Calder for which they hoped to get power from nuclear energy, and in connection with any success they might achieve in the further stages, they would be most careful to remember all that the contractors and Mr. Dick had done to help them.

Making a point about the design of the building, he said it was portal framed, and he believed that the bracing at the end of the building was taken into account as one of the main items of support against wind forces. He asked if, when designing the portal frames, it was taken into account that each portal frame would take a different load, due to the fact that the horizontal bracing at the top would deflect different distances.

Mr. F. G. Etches (Member), speaking of the framing of the reactor building, said Mr. Dick had stated that the vertical loading on the main portal frames was small. It could be seen from Fig. 5 that the legs were in fact vertical cantilevers, and as they were 140 ft. high, the moments induced in them were bound to be large. Mr. Etches asked whether a design had been tried which made use of a concrete structure divided into two cells approximately 100 ft. square, with side stiffening; in other words, a double-celled vertical box.

There would seem to be another advantage in such a design, as it would avoid high concentration of moment on the foundation raft, which was founded on a subsoil of only moderate strength.

It was interesting to hear, he added, that a 1 : 2 : 4 concrete mix was still usefully employed in this nuclear age.

Mr. B. L. Clark (Associate-Member) asked what was the shear value of the soil, because 3 tons per square foot was a most useful loading and much used value by engineers generally, even when extensive tests are not carried out, and such assessment is made by experience and visual examination. It would be interesting to know what type of material the structure was resting upon in relation to this bearing value.

Mr. J. M. Laing took the opportunity to thank Mr. Dick for the excellent way in which he had pr sented the problems involved in the design of t complicated structure of the nuclear reactor building Such complexities were not realised by many of thc who were working on the site. He also thanked M Dick for the manner in which he and his staff ha co-operated with everybody on the site, no matt by whom they were employed; the team spirit the could not have been bettered.

Mr. Alan Price added his expression of appreci tion of the unfailing help which the Contractors o the site had received from Mr. Dick. His firm we the Sub-Contractors who had built the chimneys.

Somebody had remarked that he was on the si during the only fine day of the year. That wa perhaps saying too much, but it was very nearly true and the fact that the job was completed so near t the scheduled time, in spite of all the alterations, an in spite of the conditions that prevailed, just showe what could be done when the Building Owner, th Designer, and the Contractors, were all on the sam end of the rope.

Mr. Dick, replying to Mr. Smith, said the deflectio of the roof girder and the gable end was taken int account in assessing the stresses in the portals.

He would like more time to think about Mr. Etche suggestion concerning the double cell structure there were so many factors governing the decision a the time it was made and there was a very strong case for what had been decided, and it was very difficult afterwards to assemble all the factors which were taken into account.

He said he had not at the moment the informatio concerning the shear value of the soil, mentioned b Mr. Clark, but he would give it in writing. Actua tests of the settlement on the base of clay lenses abou 5 ft. thick showed an overall settlement over 10 year of about $1\frac{3}{4}$ in. (Shear value of clay at relevant pressur is 25—30 lbs./sq. ins.)

In thanking Mr. Creasy for his remarks, Mr. Dick said that undoubtedly there was a hue and cry con cerning the business. A lot of the romance went ou of the job after one had left it, which he felt was a pity The criticism had been made that he had under-state the case; but short of writing biographies of th individuals who had taken part in the work it was very difficult to do otherwise.

Finally, he expressed his thanks to Mr. Laing and Mr. Price for their very kind remarks about thei association with the work, an association which he and his colleagues had also very much enjoyed, and he thanked the meeting for the manner in which they had received his Paper.

The meeting ended with a further expression o thanks by the President for the very pleasant and able way in which Mr. Dick had answered the very tricky questions that had been put.

Examples of Precast Ferro-Concrete Construction in France*

Discussion on Monsieur N. Esquillan's Paper

Mr. W. E. Blizard, B.Sc., M.I.C.E., M.I.W.E. President of the British Section of the Société, was in the chair and Dr. S. B. Hamilton, M.Sc., B.Sc.(Eng.), M.I.C.E., President of the Institution, accompanied him on the platform.

The Chairman, introducing M. Esquillan, said he had studied engineering at the Ecole des Arts et Metiers, where he had graduated with honours in 1922. A year later he had joined the contracting firm of the Entreprises Boussiron and had become its Technical Director. In 1933 he designed, with M. Boussiron, the bridge of La Roche Guyon, between Paris and Rouen, over the river Seine. With a span of 528 ft., that bridge held the world record for ferro-concrete road bridges with suspended deck until its destruction during the war in 1940.

He had been responsible for the design and construction of numerous ferro-concrete bridges in France and in North Africa, as, for instance, the Condette bridge in the Pyrenees. With a span of 364 ft. it had held since 1943 the world record in ferro-concrete bowstrings. The Viaduct of the Mediterranée at Chass, over the river Rhone, with a span of 406 ft. had held the world record since 1950 in ferro-concrete double track railway bridges.

M. Esquillan had made a particular study of thin shells with large spans, which had led to the design and construction of the hangars at Marignane airport, near Marseilles. With a span of 333 ft. they held the world record in the span of subtended thin arches.

Amongst his many other activities M. Esquillan was a Member of the European Committee for Ferro-Concrete; a Member of the Commission on Technical Research of the French Contractors' Association; a Member of the Committee for Ferro-Concrete of the Bureau Securitas; and a Member of the Commission for Ferro-Concrete of the Civil Engineering and Public Works Laboratories in Paris.

During the period 1944/45 he had drafted the regulations concerning the effect of snow and wind pressure on buildings, namely, the rules NV 46 of the French Ministry of Reconstruction and Town Planning. He was also the President of the Permanent Technical Commission for Research into the effects of snow and wind pressure on buildings.

He was also a Member of Council of the Société des Ingénieurs Civils de France, and in 1953 was awarded the Colomber Prize for his work on the hangars at Marignane.

In 1950 he was decorated with the Order of the Legion of Honour.

That, said the Chairman, was an extraordinary record, and it was indeed a great pleasure to introduce him to the meeting.

M. Esquillan, who was warmly applauded, first thanked the British Section of the Société des Ingénieurs Civils de France and the Institution of Structural Engineers for their kind invitation to join them at their meeting. It was indeed an honour, he said, to be given the opportunity to present a lecture to such a distinguished gathering.

Expressing his appreciation of the courtesy which had been extended to engineers in France and which had helped so much to foster the good relations that bound engineers of both countries, he said that on that day, 11th November, so full of common memories, that bond would be felt even more deeply.

He congratulated Mr. Howard, the Honorary Secretary of the British Section of the Société, on his contribution to that great cause. M. Esquillan then expressed his thanks and appreciation to Mr. Peter Gerard for all the trouble he had taken to translate the paper and to read it to the meeting.

The Chairman said how much the British Section of the Société was indebted to Mr. Peter Gerard for the work he had done in the translation of the paper; he had had discussions with M. Esquillan from time to time, and only as recently as the previous week he had been to Paris to get into his mind the details.

Mr. Peter Gerard then presented the paper and discussed the slides which illustrated the works described therein.

He also exhibited a film showing the construction of the Marignane hangars. The commentary of the film was in English and with a background of local folk-lore music.

Vote of Thanks

The Chairman, proposing a vote of thanks to M. Esquillan for his most wonderful paper and photographs and film, said it must have been a very heartfelt moment when the roofs of the hangars at Marignane were completed and the flag was flown, as depicted in the film. One must certainly congratulate him on the works he had carried out, and it was extraordinarily good of him to come to this country himself to add to the enjoyment of a very good evening. He thanked M. Esquillan very much on behalf of the Society.

* *Read before a Joint Meeting of the Institution of Structural Engineers and the British Section of the Societe des Ingénieurs Civils de France at 11, Upper Belgrave Street, London, S.W.1, on the 11th November, 1954. Published in "The Structural Engineer," Vol. XXXII, No. 11, pp. 304-311. (Nov. 1954).*

DR. HAMILTON, seconding, said it was a thrilling experience to see films and illustrations of the erection of such remarkable structures and gratifying to note that the famous firm of Boussiron were still taking a lead in the class of work in which they had always been pioneers and which they had carried to a successful conclusion on so many occasions.

He seconded the vote of thanks with very much pleasure indeed.

(The vote of thanks was accorded with enthusiasm).

Discussion

MR. B. A. E. HILEY, first complimented M. Esquillan on the wonderful way in which the paper had been prepared and illustrated, and Mr. Peter Gerard on his very fine presentation. Having heard that presentation and seen the illustrations, one could puzzle over the details quietly. It was not the kind of paper that enabled one to raise technical questions at once.

It could not be claimed by Messrs. Boussiron and M. Esquillan, he continued, that raising the 4,000 ton load of the 333 ft. span shell roofs to the hangars at Marignane was unique. A similar system of lifting great loads had been used before, and Mr. Hiley recalled that in the case of the Britannia steel tubular rail bridge by Rolt Stephenson over the Menai Straits, built in 1850, the tubular bridge girders were lifted into position on pontoons, and packing pieces placed in the grooves of the piers as the structure was lifted by the tides, the operation being repeated until the permanent level of the bridge was attained.

Again, in 1930, in the case of a bridge at Elorn-Plougastel in France, the temporary shuttering was lifted over the River Elorn in a somewhat similar manner. The three spans were each of 590 ft., and the operation was fully described by Mr. T. J. Gueritte in the paper read to this Institution, on 23rd April, 1931. More recently, his colleague, the late Mr. M. E. Gerard had to deal with a coal bunker of 5,000 tons when loaded and 1,250 tons without coal. The foundations had 'given' and he corrected this by raising one end to put the structure on an even keel to the extent of 7 ft. Again a similar method was employed.

The structures which had been described and illustrated in the paper, concluded Mr. Hiley, brought into play all the modern techniques—pre-stressed concrete, de-watering of the concrete and so on—which made the paper something which no doubt many of his fellow members, and certainly himself, would be thinking over for some time to come. They had been very much interested in both the works covered by M. Esquillan's paper.

Replying on behalf of M. Esquillan, Mr. Peter Gerard agreed that the method illustrated was not new, and he wished to draw attention to the bridge at Billancourt (Paris) which was built originally as a steel bridge and was subsequently strengthened in reinforced concrete without interrupting the traffic.

In the case of the roof of the Marignane hangars, he said the weight of 4,000 tons related to one span only, so that the total weight of the two spans was 8,000 tons.

MR. R. C. S. WALTERS, referring to the lifting operations at Marignane airport, said he understood there were 14 jacks, one under each column, and asked whether the subsoil had to be treated in preliminary way to take the pressure. He had gat ered that it was of rock; but the area was large and wondered whether the rock was uniform all over t area or whether it varied from place to place, and it did vary, had it to be treated specially?

MR. GERARD replied that the reaction was take not on the ground, but 15 ft. below ground, whe the foundations were laid on very hard subsoils, a the reaction at all times was lower than the lo bearing capacity of those foundations. The reactic was taken at the top of the base of the columns, a eventually on the lower foundations.

The subsoil was not rock, but was a material call "Safre" a local name, which is very hard and h good load bearing properties.

MR. WALTERS asked if its load bearing capaci was uniform.

MR. GERARD said that it was and there was n uneven settlement at all. The ground was so ha that a pneumatic drill had to be used to break into i

MR. H. KAYLOR, referring to the Boulogne Mari Landing Stage, asked if there were any special reaso for the use of prefabricated elements, which were well illustrated, and why advantage was not taken linking up the main beams. It would seem from t architect's drawings and from the other illustratio that the main beams were independent, continuo beams and that the secondary beams were seated c them, and were tied to the deck. He asked if t main beams could not have been made monolith with the deck; and it seemed to him that if that ha been done there might also have been a gain in hea room.

MR. GERARD replied that the saving of time an simplicity of execution were important.

MR. KAYLOR felt that, nevertheless, they might hav been put in place and a junction made, in the sam way as in the other example at Boulogne.

MR. GERARD agreed that that could have been don and, indeed, it was considered, but it was decided tha the units would then have been too long and to cumbersome to manoeuvre. Inasmuch as the acce was on speed, it was thought best to split up t prefabricated elements rather than to have one gre beam monolithic with the decking.

MR. A. GOLDSTEIN said the most striking aspects the work on the Marignane airport hangars were t boldness of conception and brilliance of executio He asked if the author could give a little more in formation about the design.

Noting that a fifth scale model was built, he sai that when one considered a fifth scale model on thought of something small; but in this instance appeared to have been about 65 ft. long, the she itself being just over 1 cm. ($\frac{1}{2}$ in.) thick. He aske whether any special measures were adopted to produc the model, and said he used the word "model" some what diffidently, because its size made the ter "structure" more appropriate.

Next he asked how and where the strains wer measured, and whether electrical or mechanica gauges were used.

Asking for further information on the genera stress distribution in the system, Mr. Goldstein sough

t know whether the stresses produced were such as ould be expected from an arch or whether the shell ffect was the more pronounced. He was thinking the span to width ratio of over 10 and said one ould expect the system to behave more or less as n arch, in the same way as if the system were flat ne would expect it to behave more or less as a beam.

Mr. Gerard said that the model was actually built f the size indicated, being 65 ft. long and the thickness f the shell had to be $\frac{1}{2}$ in. to comply with the rules of militude.

It was built during the war when testing facilities ere not readily available and so precluded the use f a smaller scale model. Acoustical strain gauges ere applied, and the deflections in the angles on all des of the building ascertained, then compared with he calculation moments.

Since the war, these results had been confirmed by pplying various other methods.

The system worked strictly as an arch.

Mr. Goldstein then asked if the transverse stresses vere very small.

Mr. Gerard replied that they were very small.

Mr. C. G. Thirlwall asked what additional reinforcement was provided in the columns of the Marignane airport hangars.

Mr. Gerard replied that the columns were always under compression, and the blocks forming these columns were provided with relatively little reinforcement.

Mr. Thirlwall asked if the transverse reinforcement was carried round ultimately to the closure panel.

Mr. Gerard replied that the stub bars were bent to meet the reinforcement and make a junction. The space was filled in afterwards and four $1\frac{1}{4}$ in. diameter threaded rods ensured a liaison between one element and the next. Grout was injected through small openings provided at the side of the blocks, but there was no grout between the blocks which were placed one above the other.

The blocks were cast on the ground and numbered. When the time came for them to be placed in position one above the other the order followed numerically the one which had been used to cast them originally, so that it appeared as if the blocks had been cast in situ.

The Chairman, at the conclusion of the discussion, again thanked M. Esquillan and Mr. Peter Gerard for the great trouble they had taken in preparing and presenting the paper and for having answered the questions raised.

And finally he voiced thanks to the Institution of Structural Engineers for having arranged the joint meeting and for having published the paper in "The Structural Engineer."

Book Reviews

(*continued from page 149*)

Advanced Mechanics of Materials, by F. B. Seely and J. O. Smith. (New York: Wiley, London: Chapman and Hall). 2nd Edition, 1952. 680 pp., 362 illustrations; 9 in. × $5\frac{3}{4}$ in., 68/-.

The first edition of this book was published in 1931 written by Professor Seely of Illinois who stated the primary purpose to be to obtain the relationships between the loads acting on members and the quantities associated with the failure of members; stress, deflection, yielding and fracture.

In the twenty years between the first and this second edition, as the Authors state, a greater need has developed in engineering design for an understanding of the advanced topics dealt with.

The contents are divided into six parts, namely:

(i) Preliminary considerations of stress formulas and theories of failure.

(ii) Members subject to static loads; dealing with shear center, unsymmetrical bending, curved beams, beams on continuous elastic support, flat plates, torsion on non-circular cross section, thick cylinders and contact stresses.

(iii) Stress concentration.

(iv) Energy methods for simple and for statically indeterminate members.

(v) Influence of small inelastic strains.

(vi) Instability and Buckling.

There are three Appendices dealing respectively with the mathematical theory of elasticity, the soapfilm analogy for torsion, and the properties of an area.

From the structural engineer's standpoint the chapters in parts (ii) (iv) and (vi) contain much of interest and value. The principles and methods dealt with are illustrated by problems worked out in the text and others offered for solution, whilst references are given for further study.

The book is well written and printed on good paper with clear and neat diagrams.

A. A. F.

"Steel Reinforcement, Cutting, Bending and Fixing," by L. A. Disney. (London: Concrete Publications, 1954.) 78 pp., 100 illustrations, 6/-.

This book is intended for draughtsmen and detailers in preparing bending schedules for steel reinforcement, and also for builders and men on the site engaged on cutting, bending, and fixing steel in reinforced concrete structures. The office work is dealt with in information on ordering the steel and examples of detailed drawings and bending schedules with notes on their preparation. There are full explanations of how to read working drawings, and chapters on practical methods of cutting, bending, and fixing the steel. An introductory chapter explains the properties of reinforced concrete.

Institution Notices and Proceedings

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ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1., on Thursday, 24th March, 1955, at 5.55 p.m., Dr. S. B. Hamilton, M.Sc., B.Sc.(Eng.), A.R.C.S., M.I.C.E., M.I.Struct.E. (President) in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated, below, should be referred to when consulting the Year Book for evidence of membership.

Students

Allaway, Godfrey Walter, of Stockton-on-Tees, Co. Durham.
Condliffe, Sidney John, of Birmingham.
Lamb, Robert, of Liverpool.
Loh Peng Hoong, of Ipoh, Perak, Malaya.
Okafor, Daniel Okeke, of London.
Pearce, Derek, of Worthing, Sussex.
Sidoryk, Tadeusz, of London.

Graduates

Abbott, Eric Norman Vincent, B.Sc.(Eng.) London, of Welling, Kent.
Alexander, Kunchatil Chacko, B.Sc.(Eng.) Travancore, of Birmingham.
Banerjee, Binoy Krishna, B.E. Calcutta, of Calcutta, India.
Banerjee, Butto Krishna, B.E.(Civil), B.Sc. Calcutta, of Stafford.
Blackledge, George Fort, B.Sc.(Tech.) Manchester, of Richmond, Surrey.
Broomhead, Graham, of Cheddar, Somerset.
Brown, Walter, A.M.I.C.E., A.M.I.Mun.E., of Gildersome, Nr. Leeds.
Coombs, Bram Stewart, of Worthing, Sussex.
Dickens, William Richmond, B.Sc.(Eng.) London, of Newport, Mon.
Evans, Arthur Thomas, of London.
Gad, Vijaykant Motiram, B.E.(Civil) Bombay, of Bombay, India.
Heron, James Alexander Stewart, of Johannesburg, South Africa.
James, Stanley Francis, of London.
Khan, Khursid Ahmad, B.A., B.Sc.(Eng.) Punjab, of Lahore, Pakistan.
Khurana, Manohar Lall, B.E.(Civil) Bombay, of Ferozepore Cantt., India.
Latawiec, Tadeusz Josef, of London.
Lozinski, Artur Jerzy, of London.
Maciag, Tadeusz, of London.
Pearson, Frank George, of Birmingham.
Pieniazek, Jozef Bronislaw, of London.
Rabalski, Jerzy Karol, of High Wycombe, Bucks.
Sanders, Richard Watkinson, B.Sc.(Civil) Bristol, of London.
Shuttleworth, Bernard Thomas, of Bolton, Lancs.
Storey, John Douglas, of Chislehurst, Kent.
Strasburger, Jerzy, of London.
Wallis, Ian Morley, of Esher, Surrey.
Whitlow, Roy (L/Cpl.), of RHQ Troop, 22 Field Engineer Regiment, M.E.L.F. 1.
Yates, John Lomax, of Liverpool.

Associate-Members

Charlesworth, Herbert John, of Bankstown, N.S.W. Australia.
Clay, Geoffrey Arthur Vernon, A.M.I.C.E., of Leeds.
Cole, Ernest Harold, of Kingston-on-Thames, Surrey.
Cornish, Richard Leslie, of Hampton, Middlesex.
De Courcy, John Wray, B.E.(N.U.I.), of Dublin, Ireland.
Gifford, Edwin William Henry, B.Sc.(Eng.) London, A.M.I.C.E., of Southampton.
Guinan-Blaney, John Henry, of Carshalton, Surrey.
Heeley, William Harold, of York.
Jabbar Khan, Saadat Abdul, B.E.(Civil) Madras, D.I.C., of London.
Johnson, Leslie Gordon, M.A.(Cantab.), of Dewsbury, Yorkshire.
McClung, Kenneth Gilman, B.Sc.(Tech.) Manchester, of Manchester.
McNaughton, Finlay Malcolm, A.M.I.C.E., A.M.I.Mun.E., of Bolton, Lancs.
Metter, Louis William, B.Sc.(Eng.), Witwatersrand, of Johannesburg, South Africa.
Perry, Anthony Percy, B.A.(Cantab.), of Northwood, Middlesex.
Ridley, Philip George, B.Sc.(Eng.) London, of London.
Small, Arthur John Campbell, A.M.I.C.E., A.M.I.Mun.E., of Dar-es-Salaam, Tanganyika.
Smith, Brian Groome, B.Sc.(Eng.) London, A.M.I.C.E., of Christchurch, Hants.
Smullen, Herman, of London.
Thomas, David, of Greenford, Middlesex.
Wilson, Fred., B.Sc.(Eng.) London, of Nottingham.

Members

Andrews, George Richard Rowland, of London.
Galea, Victor Robert, M.A. Oxford, B.Sc.(Malta) B.E. & A.(Malta), of St. Julian's, Malta.

Transfers

Students to Graduates

Brownlow, John Hamilton, of Manchester.
Chung Cheng Chek, of Singapore.
Cook, Alun Morgan, of Rhymney, Mon.
Feakes, Gerald William Ernest, of Reading, Berks.
Fergusson, James Colin, of Benoni, Transvaal, South Africa.
Gilder, Peter James, of London.
Lawrence, Alan Albert, of Weybridge, Surrey.
Manthe, Peter Eric, of Queenstown, South Africa.
Nadarasa, Mailvaganam, of London.
Snaddon, Robert Angus Anderson, of Johannesburg, South Africa.
Stewart, Alan Hugh Atkinson, of Bothwell, Lanarkshire.
Stuart, Peter Leslie, of London.
Van Essen, Albert, of Johannesburg, South Africa.

Graduates to Associate-Members

Abbs, Ronald Samuel, of Irlam, Nr. Manchester.
Boggon, William George, B.Sc.(Civil) Bristol, of Stanley, Co. Durham.
Bowles, Anthony John Hastings, B.Sc.(Eng.) London, of Kingston-upon-Thames, Surrey.
Bramwell, Henry Leslie, of Birmingham.
Britt, Geoffrey Brian, M.A.(Cantab.), of Birmingham.
Cooper, Bryan Walton, B.Sc. Birmingham, D.I.C., of Faversham, Kent.
Davies, John Duncan, B.Sc.(Eng.) London, of Swansea, South Wales.
Day, John Gordon, B.E.(Civil) Queensland, of Brisbane, Queensland, Australia.
Dilks, John Roxby, B.Sc.(Civil) Durham, of Gosforth, Newcastle upon Tyne.
Gibson, Peter Lovatt, of Liverpool.
Grice, John Robert, B.Sc.(Tech.) Manchester, of Wythenshawe, Manchester.
Hudson, Hilbert Leslie, of Clarendon, Jamaica, B.W.I.
Ironmonger, Albert Edward, of Rainham, Essex.
Kiley, Gerard, of London.
Korn, Heinz Guenter, B.Sc.(Eng.) London, of Harlow, Essex.
Little, Derek Sidney, A.M.I.C.E., of Romford, Essex.
Luker, Colin Rae, of London.
Mairs, James Girvan, B.Sc.(Civil) Glasgow, of Glasgow.
Maxted, Alan John, A.M.I.C.E., of Kettering, Northants.
Mead, Frank Hillson, B.Sc.(Eng.) London, of Luton, Bedfordshire.
Meineke, Cyril Christopher, of Johannesburg, South Africa.
Northgreaves, Kenneth Roy, of Surbiton, Surrey.
Partridge, Malcolm Harold, of Walsall, Staffordshire.
Pun Yin Keung, B.Sc., D.I.C., of London.
Rodger, William, of Greenock, Renfrewshire.
Seager, Michael William, of Chesterfield, Derbyshire.
Taunt, David Anthony, B.Sc.(Eng.) London, A.M.I.C.E., of Ashtead, Surrey.

Associate-Members to Members

Bowen, Frank Maurice, M.I.C.E., Assoc.I.Mech.E., of Petts Wood, Kent.
Johncock, Dennis Frank, of New Malden, Surrey.
Moore, Harold William, of London.

Members to Retired Members

Simons, Charles Gordon, of Bridge of Allan, Stirlingshire.
White, John Maurice, of Douglas, Isle of Man.

Obituary

The Council regret to announce the deaths of John Charles Telford, O.B.E. (Retired Member), Ernest Thomas Bevan (Associate), Edwin John Cain, Capt. Norman Alexander Grandage, John Lerwell Ince (Associate-Members).

Resignations

Notification was given that the Council had accepted with regret the resignations of Oscar Borer (Member), Trevor Lewis (Associate-Member), Miss Maureen Emmerson, Donald Hilton Wright (Graduates).

Examinations, January, 1955

Home and Overseas Centres

The examinations were held in January 1955 at the usual centres in Great Britain and overseas at the following centres :

Baghdad, Bombay, Brisbane, Cairo, Calcutta, Cape Town, Colombo, Cooma, Dar-es-Salaam, Delhi, Dunedin, Durban, East London (South Africa), Georgetown (British Guiana), Hong Kong, Jerusalem, Johannesburg, Kampala, Karachi, Khartoum, Kirkuk, Kisumu, Kuala Lumpur, Lahore, Madras, Miri, Montreal, Nairobi, Port Elizabeth (South Africa), Salisbury (Southern Rhodesia), Singapore, Sydney, Toronto, Wellington.

Seventy candidates took the Graduateship Examination (34 at Home centres and 36 Overseas) ; 388 candidates took the Associate-Membership Examination (316 at Home centres and 72 Overseas), making a total of 458. Of these, 32 passed the Graduateship Examination (21 at Home centres and 11 Overseas), and 72 passed the Associate-Membership Examination (64 at Home centres and 8 Overseas). The names of the successful candidates are :

Graduateship Examination

Home Centres

Basson, William Eric Stanley.
Broomhead, Graham.
Evans, Arthur Thomas.
Gilder, Peter James.
Hallows, Peter Marcus.
Higham, Roland.
James, Stanley Francis.
Ketterer, Robert W.
Latawiec, Tadeusz.
Lozinski, Artur Jerzy.
Marr, John Smith.
Nadarasa, Mailvaganam.
Rabalski, Jerzy Karol.
Shuttleworth, Bernard Thomas.
Silver, Derek Percival.
Strasburger, Jerzy.
Stuart, Peter Leslie.
Tregilgas, William Gerald.
Welford, Paul Alexander.
Yates, John Lomax.
Young, Reginald Alfred.

Overseas Centres

Chung Cheng Chek.
Fergusson, James Colin.
Fyshe, Anthony Preston.
Heron, James Alexander Stewart.
Husain, Syed Qaisar.
Manthe, Peter Eric.

MICHAEL, Felix.
SNADDON, Robert Angus Anderson.
TANDY, Robert Macmillan.
VAN ESSEN, Albert.
VERKROOST, Johan.

ASSOCIATE-MEMBERSHIP EXAMINATION

Home Centres

ABBS, Ronald Samuel.
ALLUM, Clive Robert.
BARNWELL, Derek.
BINGHAM, David Malcolm.
BOGGON, William George.
BOOTH, Alan Sutherland.
BOWLES, Anthony John Hastings.
BRAMWELL, Henry Leslie.
BRETT, Peter Ronald.
BRITT, Geoffrey Brian.
BULLOCK, Francis Hubert.
CICHONSKI, Feliks.
CLAY, Geoffrey Arthur Vernon.
COLE, Ernest Harold.
COOPER, Bryan Walton.
CORNISH, Richard Leslie.
CRAIG, Cameron.
CZOSKE, Jan Edmund.
DALE, John Granton.
DAVIES, David Charles.
DAVIES, John Duncan.
DE COURCY, John Wray.
DILKS, John Roxby.
ELLIS, William George.
FRANCIS, Rhys Hugh.
GIBSON, Peter Lovatt.
GIFFORD, Edwin William Henry.
GRICE, John Robert.
GUINAN-BLANEY, John H.
HARDY, Donald.
HAYMAN, Kenneth Maldwyn.
HEELEY, William Harold.
HIBBERT, Norman Trevor.
IRONMONGER, Albert Edward.
JABBAR-KHAN, Sandat Abdul.
JOHNSON, Leslie Gordon.
KILEY, Gerard.
KORN, Heinz Guenter.
LITTLE, Derek Sidney.
LUKER, Colin Rea.
MCCLUNG, Kenneth Gilman.
MCNAUGHTON, Finlay Malcolm.
MAIRS, James Girvan.
MAXTED, Alan John.
MEAD, Frank Hillson.
NELSON, Kenneth.
NEWTON, Michael William.
NORTHGREAVES, Kenneth Roy.
PARTRIDGE, Malcolm Harold.
PERRY, Anthony Percy.
RICHARDS, Gerald William.
RIDLEY, Philip George.
ROBINSON, William Whitfield.
RODGER, William.
SEAGER, Michael William.
SMITH, Brian Groome.
SMULLEN, Herman.
TAUNT, David Anthony.
THOMAS, David.
VASWANI, Harkrishin Pahlejrai.
WILLIAMS, Robert Raymond.
WILLINGTON, John Ernest.
WILSON, Fred.
WILSON, William Edward.

Overseas Centres.

CHARLESWORTH, Herbert John.
CURSITER, William Nisbet.
DAY, John Gordon.
MEINEKE, Cyril Christopher.
METTER, Louis William.
SMALL, Arthur John Campbell.
THOMAS, David Lewis.
WILDEN, Leonard Kenneth.

PRIZES—JANUARY 1955 EXAMINATIONS

The Council have awarded the following prizes i respect of the examinations held in January, 1955 :

ANDREWS PRIZE (For the candidate who obtains t highest aggregate of marks in the Associate-Membe ship Examination, passing in all subjects)

Philip George RIDLEY, of Ealing.

HUSBAND PRIZE (For the candidate who takes t whole of the Associate-Membership Examinatio passes in all subjects, and obtains the highest mar in the paper "Structural Engineering Design an Drawing")

Michael William SEAGER, of Chesterfield.

WALLACE PREMIUM (SENIOR) (For the candidate wh takes the whole of the Associate-Membership Examin tion, passes in all subjects, and obtains the highe marks in the paper "Theory of Structures (A vanced)")

Philip George RIDLEY, of Ealing.

WALLACE PREMIUM (JUNIOR) (For the most succes ful candidate in the Graduateship Examination, pas ing in all subjects)

Johan VERKROOST, of Southern Rhodesia.

EXAMINATIONS—JULY, 1955

The Examinations of the Institution will next b held at centres in the United Kingdom and Overse on the 12th and 13th July, 1955 (Graduateship) an the 14th and 15th July (Associate-Membership).

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upp Belgrave Street, London, S.W.1.

Thursday, May 26th, 1955

Ordinary General Meeting for the election of mem bers, 5.55 p.m.

Annual General Meeting of the Institution, 6 p.m.

Annual General Meeting of Voting Contributors t the Institution of Structural Engineers' Benevolen Fund, 6.30 p.m.

Thursday, June 23rd, 1955

Ordinary General Meeting for the election of mem bers, 5.15 p.m.

HONOURS AND AWARDS

In offering their sincere congratulations to the follow ing members on the distinctions recently conferred upo them, the Council feel they are also expressing the goo wishes of the Institution :

Baronet

Mr. L. Lister-Kaye (Member).

Order of St. Michael and St. George—C.M.G.

Mr. T. L. Bowring, O.B.E. (Member).

Order of the British Empire—C.B.E.

Mr. O. N. Arup (Member).
Mr. J. M. Fraser (Associate-Member).

Order of the British Empire—M.B.E.

Mr. F. T. Ephraums (Associate-Member).
Mr. A. Rogers (Associate-Member).

REPRESENTATION

The Council have made the following nominations of members to represent the Institution :

L.C.C. School of Building, Governing Body
Mr. Walter C. Andrews (Past President).

London Building Acts (Amendment) Act 1939, *Tribunal of Appeal*
Lt.-Colonel R. F. Galbraith (Past President).
Mr. F. M. Bowen (Member)—Deputy.

MACLACHLAN LECTURE COMPETITION

The conditions of the Maclachlan Lecture Competition have been revised to include Graduates of the Institution as well as Associate-Members who are under the age of 32 years. The Competition is held annually and the closing date for the receipt of entries for the lecture to be given in 1956 is Friday, March 30th, 1956. The subject may cover any aspect of structural engineering, and the work should be submitted as the script of a lecture which the author, if successful in the Competition, will deliver before an audience in the course of about one hour.

Full particulars of the Competition may be obtained from the Secretary of the Institution.

DRURY MEDAL AWARD

The fifth competition for the above award will take place in 1955. The subject is the design of a mobile crane.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary : envelopes to be marked in the top left-hand corner, " Drury Medal Award."

The closing date for the competition is October 1st, 1955.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say, primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution, or failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate in the presence of a Justice of the Peace or a Commissioner for Oaths.

RESEARCH AWARDS

The Council have instituted a Research Prize Fund, from which awards may be made annually to the author or joint authors of papers describing original research which they have carried out. Research awards may be made for papers read at Headquarters or in the Branches and published in the Journal, or for papers published in the Journal only without being read at an open meeting.

The assessment for such awards will be made annually, but awards will be made only to the contributors of such papers as reach a standard judged by the Literature Committee to be satisfactory.

Work submitted under this scheme must be original and may include any of the following :

(a) investigations of an experimental or analytical character ;
(b) studies of historical or statistical records ;
(c) improvements in principles or methods of construction ;
(d) research into methods of structural engineering and building, the nature and use of plant and the organisation or engineering work ;
(e) any related or combined studies which are deemed by the Literature Committee to be of a research character.

In cases where the research work described in the paper was not the work of one individual, the names of all the collaborators should be given in the paper.

Awards may take any or all of the following forms : A research medal ; a diploma ; a money prize.

Application for consideration for a research award must be made to the Secretary of the Institution, and in preparing papers for reproduction in the Journal, authors must comply with the conditions laid down for all such contributions. Particulars of these conditions may be obtained from the Secretary.

In judging research papers, the following factors will be considered :

(a) the nature of the subject and its conclusions ;
(b) the value of the paper in advancing the science and art of structural engineering ;
(c) the standard of preparation and orderly arrangement of the subject-matter.

Research papers will also be eligible for adjudication for the Institution Medals if they comply with the Regulations governing those awards.

The closing date for the receipt of applications in respect of papers published in the Journal between October, 1954, and September, 1955, is October 3rd, 1955.

LONDON GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : J. A. Pope, 53, Cranleigh Drive, Leigh-on-Sea, Essex.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

Joint Hon. Secretaries : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire ; M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford, 6, Lancs.

MIDLAND COUNTIES BRANCH

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : A. K. A. Costain, A.M.I.C.E., 134, Witherford Way, Weoley Hill, Birmingham, 29.

NORTHERN COUNTIES BRANCH

Hon. Secretary : Captain O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., " Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., " Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH WESTERN COUNTIES BRANCH

The Annual General Meeting will be postponed from Friday, 20th May to Friday, 27th May, and will be held at the Duke of Cornwall Hotel, Plymouth.

The Annual General Meeting will commence at 6 p.m. and will be followed at 7.15 p.m. by a paper entitled " The Development of Plastic Design " to be given by Professor J. F. Baker, O.B.E., M.A., D.Sc., M.I.C.E. (Member).

Joint Hon. Secretaries : E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay; C. J. Woodrow, " Elstow," Hartley Park Villas, Tavistock Road, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The Annual General Meeting of the Branch will be held at the offices of the Steel Company of Wales, Port Talbot, on Tuesday, May 3rd, 1955, at 6.30 p.m.

Hon. Secretary : K. J. Stewart, A.M.I.C.E., A.M.I.Struct.E., 15, Glanmor Road, Swansea.

WESTERN COUNTIES BRANCH

Hon. Secretary : E. Hughes, M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., P.O. Box No. 3306, Johannesburg, South Africa.

During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone : 34-1111, Ext. 257.

Natal Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co. Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

ADDITIONS TO THE LIBRARY

ALLEN, D. N. de G. *Relaxation Methods*, New York and London, 1954. Presented by Mr. H. C. Husband

American Society of Civil Engineers Transactions, Vol. 119, 1954, New York, 1954.

Architect's and Builder's Desk Diary and Building Legislative Index, 1955, London, 1955.

First Australia-New Zealand Conference on Soil Mechanics and Foundation Engineering Proceedings, Melbourne, 1954.

BAKER, J. F. *The Steel Skeleton*, Vol. I; *Elastic Behaviour and Design*, Cambridge, 1954. Presented by Mr. S. M. Iqbal.

BARDOUT, G. and BERNY, P. *La Construction des Tunnels, Galeries et Souterrains*, Paris, 1954. Presented by Mr. P. J. Gerard.

BETEILLE, P. *Resistance des Materiaux*, Vol. II, Paris, 1954. Presented by Mr. G. B. Godfrey.

BRANDT, D. J. O. *The Manufacture of Iron and Steel*, London, 1953.

British Engine Technical Report, New Series, Vol. II, Manchester, 1954. Presented by the Publishers.

British Standards Institution Annual Report, 1953-54, London, 1954.

BROOKS, C. *An Educational Adventure : A History of the Woolwich Polytechnic*, London, 1955. Presented by the Publishers.

Canadian Conference on Prestressed Concrete, 1954 *Discussions of Papers*, Toronto, 1954. Presented by Mr. J. A. Cole.

CHAMECKI, S. *As linhas de influencia nos portico continuous planos e do espaco tridimensional*, Brazil 1954. Presented by the Author.

COULTAS, H. W. *Theory of Structures*, 4th Edition London, 1955.

Deutscher Stahlbau-Verband, 1904-1954, Cologne, 195 Presented by the Publishers.

FABER, O. *Constructional Steelwork Simply Explained* 3rd Edition, London, 1954.

FRITH, P. H. *Fatigue Tests on Rolled Alloy Steel made in Electric and Open Hearth Furnaces*, London 1955. Presented by the Publishers.

GIBSON, J. E. and D. W. COOPER. *The Design of Cylindrical Shell Roofs*, London, 1954. Two copies presented by Mr. D. W. Cooper and by Mr. W. H Sneddon.

GRAY, C. S., L. E. KENT, W. A. MITCHELL and G. F GODFREY. *Steel Designer's Manual*, London, 195 Presented by Dr. E. H. Bateman.

Institution of Mechanical Engineers, Automobile Division Proceedings, 1952-53. London, 1955.

International Association for Bridge and Structura Engineering Publications, Vol. XIII, 1953; Vo XIV, 1954, Final Report of the 4th Congress Zurich, 1954.

The Ironmonger Pocket Book of Tables, 8th Edition London, 1954.

JONES, R. and E. N. GATFIELD. *Testing Concrete b an Ultrasonic Pulse Technique* : Road Researc Technical Paper, No. 34, London, 1955.

KING, J. H. G. and D. A. CRESWELL. *Soil Mechanic Related to Building*, London, 1954. Presented b Mr. C. B. Brown.

KNIGHT, B. H. *Surveying and Levelling for Students* London, 1954. Presented by Major Norman C Sidwell.

LAURSON, P. G. and W. J. COX. *Mechanics of Material* 3rd Edition, New York and London, 1954.

LIPS, E. M. H. *Engineering Metallurgy*, London, 1954 Presented by Mr. W. Shearer Smith.

MANNING, G. P. *Reinforced Concrete Arch Design* 2nd Edition, London, 1954.

MARIN, J. and SAUER, J. A. *Strength of Materials* 2nd Edition, New York, 1954. Presented by Mr D. T. Williams.

MAZZONI, A. *The Steam Vents of Tuscany and th Larderello Plant*, 2nd Edition, Bologna, 1954. Pre sented by the Author.

The Modular Catalogue, Vol. I, London, 1955.

MOLESWORTH, E. J. *Life of Sir Guilford L. Molesworth* London, 1922. Presented by Mr. E. G. Lester.

PARKER, HARRY. *Simplified Design of Structural Steel*, 2nd Edition, New York and London, 1955.

PETERSON, R. E. *Stress Concentration Design Factors* New York and London, 1953. Presented by Mr G. S. Gowland.

PIPPARD, A. J. S. and J. F. BAKER. *The Analysis o Engineering Structures*, 2nd Edition, London, 194 Presented by Mr. K. Slankiewicz-Wismiewski.

ROSS, J. *Data for Use in Structural Steel Design t Conform with the Requirements of British Standar* 449 : 1948, London, 1955.

SEELYE, E. E. *Data Book for Civil Engineers*, Vol. III *Field Practice*, 2nd Edition, New York and London 1954.

SIAND, P. MORTON. *Building : The Evolution of an Industry*, London, 1954. Presented by the Publishers.

mithsonian Institution Annual Report of the Board of Regents for the Year Ended June, 1953, Washington, 1954.

TUSSI, F. *Das Problem der grossen Spannweite* (*The Problem of the Long Span*), Zurich, 1954. Presented by the Author.

WAINGER, K. H. *Analysis of Deformation*, Vols. I & II, London, 1954. Presented by Mr. F. A. Gerard.

RQUHART, L. C., C. E. O'ROURKE and G. WINTER. *Design of Concrete Structures*, New York and London, 1954.

ANDEPITTE, D. *Het Draag vermogen van Paalfunderingen*, Brussels, 1953. Presented by the Author.

VANG, CHU-KIA. *Statically Indeterminate Structures*, New York and London, 1953. Presented by Dr. A. A. Fordham.

onference on Welded Structures Proceedings, 1953, London, 1954.

VILLIAMS, G. A. *Heating in Industry and Ventilating in Industry*, Richmond, Surrey, 1954.

OUNG, J. MCHARDY. *The Application of Light Alloys to Structural Engineering*, Richmond, Surrey, 1955.

Copies of the following papers, in respect of which Branch Prizes have been awarded, have been placed in the Library for reference :—

BROOKSBANK, F. *Designing for Welding.*

ROBERTS, A. H. K. *Some Practical Applications of Prestressed Concrete.*

NEWTON, J. A. *The Civil and Structural Engineers' Contribution towards the Reconstruction of Stapleton Road Gas Works.* (Text only.)

COOPER, S. M. *The Tacoma Narrows Bridge—Design Features and Collapse Investigations.*

BROTTON, D. M. *The Application of Relaxation Methods to the Solution of Problems in Structural Engineering.*

MALCOMSON, J. C. *Recent Developments in Prefabricated Concrete Structures.*

The following paper has been placed in the Library for reference :

BARTLETT, R. J. *Variable Moment of Inertia.*

INSTITUTION LIBRARY

Conditions of Loan from the Library

1. The Library is available to members of the Institution only. In special circumstances, facilities may be granted to non-members to use the Library, but they will not be allowed to borrow books.
2. Members of the Institution may borrow any book or pamphlet which is in general circulation, and also copies of Transactions and periodicals.
3. Rare books and current works of reference are not available on loan, but may be consulted in the Library.
4. Members may borrow books either by making personal application or by post.
5. The period of loan is a fortnight, but this period may be extended for a further fortnight provided that the book is not required by any other member, and provided that application for extension is made before the original period of loan has expired. The Institution pays all outgoing postage. The members are responsible for the return postage.
6. Loans are only made on the understanding that the borrower will replace any book or periodical that may be lost or damaged through any circumstances connected with the loan, including the period of transit to and from the Library. All publications should be returned by registered post and adequately protected against damage.
7. The Library is open from Monday to Friday, from 9.0 a.m. to 5.0 p.m. during the Session, and 9.30 to 5.0 p.m. during the Recess.
8. A Library Catalogue is not available at present.

PAPERS FOR PUBLICATION

The Literature Committee would be glad to consider offers of papers for presentation at the Institution or for publication in the Journal.

The following is a summary of the Committee's requirements relating to articles and papers : a copy of the full conditions may be obtained from the Secretary.

(1) Articles must be of an appropriate character, having a bearing upon structural engineering or upon some kindred scientific or constructional subject, and must be approved by the Literature Committee. A short title is an advantage.

(2) Contributions must be original either in subject-matter or in presentation. Articles which have already been published or have been read to other organised bodies, or are carelessly prepared, will not be accepted for publication.

(3) The style of writing will necessarily vary with the individual, but authors are requested to write as plainly and simply as their subject will allow. Papers should be written in the third person.

(4) Where the subject allows, a brief introduction or synopsis should state clearly the purpose and scope of the paper or article, and the author's conclusions or recommendations should be summarised at the end of the paper.

In order to facilitate the indexing of articles for reference, the author will be required in addition to prepare a short precis not exceeding 25 words for inclusion under the title of the paper on the contents page of the Journal.

(5) Illustrations are desirable where they assist in explaining the context or are fundamental to the subject. They should not be used if unnecessary for these purposes. Illustrations may be either line drawings or photographs.

(6) Line drawings must be specially prepared for reproduction on smooth white paper, or clear tracing paper, with heavy main lines and large clear lettering drawn in Indian ink with a mapping pen. Alternatively, the author may submit drawings on one sheet of paper with the relevant lettering on a cover sheet of tracing paper.

The printed page of THE STRUCTURAL ENGINEER is 7 in. wide by 10 in. deep. The drawings, where practicable, should be prepared not larger than twice this size with a view to half-scale reproduction. Unavoidably large drawings which require reduction to one-third size or less, must be specially heavy and with proportionately large lettering for clear reproduction. Ordinary working drawings are not satisfactory.

(7) Where photographs are submitted they should be printed *black on glossy paper.*

(8) MS. typewritten in double spacing should be submitted in duplicate.

Brevity is an advantage and papers should not normally exceed 7,500 words in length.

REPRINTS FROM " THE STRUCTURAL ENGINEER "

Authors who wish to obtain reprints of their papers published in " The Structural Engineer " should notify the Secretary of the Institution not later than the 15th of the month in which the paper is published.

EXAMINATIONS

Preparation for the Examinations of the Institution by Attendance at Technical Colleges

A candidate for Graduateship or Associate-Membership may be able to attend a technical college ; these notes are intended to guide him in choosing the most suitable instruction.

Preparation for the Graduateship Examination

Technical Colleges offer :

(a) Full-time courses for degrees or Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At technical colleges courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

Preparation for the Associate-Membership Examination

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering ; these cover only part of the requirements for the Associate-Membership Examination.

Colleges in List "A" provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

List "A"

Bath Technical College.
Belfast College of Technology.
Birmingham College of Technology.
Bolton Municipal Technical College.
Bradford Technical College.
Bridgend Technical College.
Chesterfield College of Technology.
Derby Technical College.
Dudley and Staffordshire Technical College.
Glasgow Royal Technical College.
City of Liverpool College of Building.
L.C.C. Brixton School of Building, S.W.4.
L.C.C. Hammersmith School of Building and Art and Crafts, W.12.
Manchester College of Technology.
Middlesbrough, Constantine Technical College.
Nottingham and District Technical College.
Salford, Royal Technical College.
South-East London Technical College, Lewisham Way, S.E.4.
South-West Essex Technical College, Walthamstow E.17.
Stafford, County Technical College.
Stockport College for Further Education.
Twickenham Technical College.
Willesden Technical College, N.W.10.

Colleges in List " B " provide instruction in Theory of Structures from which the student may reach Associate Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete.

List " B "

Brighton Technical College.
Cardiff Technical College.
Edinburgh, Heriot-Watt College.
Huddersfield Technical College.
Leeds College of Technology.
London, Battersea Polytechnic, S.W.11.
London, Northampton Polytechnic, E.C.1.
L.C.C. Westminster Technical College, S.W.1.
Newcastle-upon-Tyne, Rutherford College of Technology.
Plymouth and Devonport Technical College.
Preston, Harris Institute.
Rotherham College of Technology.
Wigan Mining and Technical College.
Woolwich Polytechnic, S.E.18.
West Ham College of Technology.

Students are advised to take the organised courses in Structural Engineering where these are available.

YEAR BOOK AND LIST OF MEMBERS

The Year Book and List of Members for 1955 will go to press in July, for publication in October, when a copy will be sent to all members.

Members are requested to inform the Secretary of any alterations in titles, degrees or addresses, which have not already been notified, by July 4th, in order that such amendments may be included in the new edition.

Energy Approximations Applied to Problems of Structural Equilibrium, Stability and Vibrations

By P. B. Morice, B.Sc., Ph.D.

Summary

The paper discusses the general energy basis of structural problems of equilibrium, stability and vibrations making use of Hamilton's principle of least action as a starting point. The forms which the energy takes in the various cases are shown and the analytical method of solution is then followed. This is compared with the functional approximation methods of Rayleigh-Ritz, Lagrange and Galerkin. The useful property of orthogonality is briefly discussed. A number of examples are then given which demonstrate the use of the above methods to some simple problems of structural mechanics.

Introduction

The earliest investigations of the use of functional approximations for the solution of differential equations seem to be those of Lagrange, but undoubtedly their application to mechanical problems was greatly influenced by Lord Rayleigh's introduction of a very simple approximate method based on energy considerations and the later researches of Ritz. Further work has been done in recent years but has to some extent been overshadowed by the introduction of numerical methods and the use of modern high speed computers. Nevertheless, the earlier methods have many advantages over the numerical methods amongst which is their simplicity of application. In general the methods have been used where precise solutions have not been found although they are often of use where the precise solution involves the evaluation of complex transcendental equations or slowly convergent series. In a number of cases the procedures can be extended to provide exact solutions.

The present paper is concerned with structural problems and the "principle of least action," or "Hamilton's principle," is used as a starting point. A general approach is adopted which embraces static equilibrium, stability and vibrations.

The procedures discussed are the Rayleigh-Ritz method, the Lagrangian multiplier method and Galerkin's method. The Rayleigh-Ritz method is probably the best known and in general the most straightforward in application. The procedure is usually outlined in modern text books on mechanics, e.g. Wang.[1]

The Lagrangian multiplier extension of the Rayleigh-Ritz method seems only recently to have been applied to structural problems, although it is in fact chronologically the earlier method, the latter being a particular case. Its application in some cases of stability has been given by Budiansky.[2] Galerkin's method and its use in a number of elasticity problems has been discussed by Duncan.[3]

In the present paper the development and description of the methods is followed by a number of examples of their use. These examples have purposely been made simple in order to avoid the analysis being confused with detail.

GENERAL ENERGY BASIS

Consider a vibrating elastic structure in which there is no periodic applied force increasing the energy of the system and no damping forces absorbing energy. It is further assumed that the supports do no work during a deformation. The system is then a conservative system and the total energy must remain constant.

The energy of the system is composed of three parts, the strain energy U of the structure due to its deformation from the stress-free state, the change in potential energy V of the applied loads, also due to the deformation, and the kinetic energy T of the structure due to its motion.

As a starting point for the study of the motion use is made of Hamilton's principle which states that the action

$$\int_{t_1}^{t_2} \Big\{ T - (U+V) \Big\} dt \quad . \quad . \quad . \quad . \quad (1)$$

is a minimum.

In order to proceed further it is necessary to investigate the forms of U, V and T. It is assumed that the total deformation w of the structure at any instant is the sum of two deformations $w = w_1 + w_2$ where $w_1 = \psi_1 (x, y, z)$ is the deformation of the structure in static equilibrium and $w_2 = \psi_2 (x, y, z)\ \psi_3 (t)$ is the deformation due to motion.

The strain energy of the structure is an integral quadratic function of the deformation and thus

$$U = \Phi(w^2) = \Phi(w_1{}^2) + \Phi(w_2{}^2) + 2\Phi(w_1 w_2) \quad (2)$$

Furthermore, if it is assumed that the structure is composed of linear elastic material, the stresses $\widehat{\eta\xi}$ and the strains $e_{\eta\xi}$ will each be linear functions of the deformation w and therefore the strain energy can be written in terms of the stresses and strains

$$U = \Phi(w^2) = \tfrac{1}{2} \int_v \widehat{\eta\xi}\, e_{\pi\xi}\, dv \quad . \quad . \quad . \quad . \quad (3)$$

where η and ξ are summed over the spacial co-ordinates; the cartesians x, y, z or the cylindrical polars r, θ, z, etc.

The change in potential energy is the gain in energy of the loads during the deformation w. In the case of static equilibrium between a structure and its applied loads P_i this is

$$V_1 = -\sum_i P_i \, w_{1i}$$

where w_{1i} are the displacements at and in the directions of P_i due to the deformation w_1. In the subsequent deformation w_2 with displacements w_{2i} at and in the directions of the loads it is assumed that the values of the loads are independent of the deformation and the potential energy change is thus

$$V_2 = -\sum_i P_i \, w_{2i}$$

The total change in potential energy is therefore the sum of V_1 and V_2

$$V = V_1 + V_2 = -\sum_i P_i \, (w_{1i} + w_{2i}) \quad . \quad (4)$$

The kinetic energy of the structure is the product of half the mass and the velocity squared

$$T = \sum_j \tfrac{1}{2} \frac{m_j}{g} \left(\frac{\partial w}{\partial t} \right)_j^2$$

Since w_1 is independent of the time this becomes

$$T = \sum_j \tfrac{1}{2} \frac{m_j}{g} \left(\frac{\partial w_2}{\partial t} \right)_j^2 \quad . \quad . \quad . \quad . \quad . \quad (5)$$

The Hamiltonian integral may, therefore, be rewritten with the expressions for U, V and T giving

$$\int_{t_1}^{t_2} \left\{ \sum_j \tfrac{1}{2} \frac{m_j}{g} \left(\frac{\partial w_2}{\partial t} \right)_j^2 - \Phi(w_1{}^2) - \Phi(w_2{}^2) - 2\Phi(w_1 w_2) + \sum_i P_i (w_{1i} + w_{2i}) \right\} dt \quad (6)$$

Various simplifying relationships within the integral (6) may now be considered for various structural conditions.

The first is that in a deformation of static equilibrium the terms involving w_2 may be set equal to zero and Hamilton's principle of least action reduces to the "Principle of Minimum Potential Energy" and states that

$$\sum_i P_i \, w_{1i} - \Phi(w_1{}^2) \quad . \quad . \quad . \quad . \quad (7)$$

is to be minimized, which enables the deformation w_1 to be determined. The time integral has been dropped since the problem is one of statics. This principle is not dependent upon a linear relationship between loads and displacements. Williams[4] has discussed its use in structural problems.

The second point to be noted is that in the subsequent deformation w_2 from the position wi of static equilibrium the minimization is unaffected by the terms involving w_1 only and these may therefore be omitted. If the deformation w_2 is orthogonal to the deformation w_1 then the term $\Phi(w_1 w_2)$ disappears. Such conditions characterize stability problems. In these cases all the displacements w_{2i} will be quadratic functions of the deformation w_2. The Hamiltonian integral now becomes

$$\int_{t_1}^{t_2} \left\{ \sum_j \tfrac{1}{2} \frac{m_j}{g} \left(\frac{\partial w_2}{\partial t} \right)_j^2 - \Phi(w_2{}^2) + \sum_i P_i w_{2i} \right\} dt \quad . \quad .$$

If, on the other hand, the displacements w_{1i} and w_{2i} are linearly related to their corresponding deformations w_1 and w_2 then it follows that

$$\sum_i P_i \, w_{2i} = 2 \, \Phi \, (w_1 w_2)$$

Such problems are ones of simple vibrations and the Hamiltonian integral becomes

$$\int_{t_1}^{t_2} \left\{ \sum_j \tfrac{1}{2} \frac{m_j}{g} \left(\frac{\partial w_2}{\partial t} \right)_j^2 - \Phi(w_2{}^2) \right\} dt \quad . \quad .$$

If the displacements w_{1i} and w_{2i} are not all linearly related to their deformations, and the deformations w_1 and w_2 are not orthogonal the problem is one of non-linear vibrations. Such a problem would be that of the oscillation of a beam with combined side load and end load. Problems of this nature will not be discussed here.

It will be assumed that in the cases of vibration to be considered the time dependent function $\psi_3(t)$ is the harmonic $\sin 2\pi\omega t$, where ω is the frequency of oscillation. The two cases of vibration to be studied are those represented by integrals (8) and (9) where it is now seen that the time dependent function will appear in all terms as $\sin^2 2\pi\omega t$ or $4\pi^2\omega^2 \cos^2 2\pi\omega t$. If the integration is carried out over the period of one cycle $\frac{1}{\omega}$, then

$$\int_0^{\frac{1}{\omega}} \sin^2 2\pi\omega t \, dt = \int_0^{\frac{1}{\omega}} \cos^2 2\pi\omega t \, dt = \frac{1}{2\omega}$$

Thus the integration with respect to time merely has the effect of introducing a constant multiplier which is the same for each term and since this cannot affect the determination of the minimum it may be discarded. The expressions which require to be minimized for the three cases investigated are therefore the following in which the strain energy has now been written in terms of the stresses and strains. It is to be noted that the

resses and strains are those due to w_1 in (10) and those ue to w_2 in (11) and (12).

tatic equilibrium

$$\sum_i P_i w_{11} - \tfrac{1}{2}\int_v \widehat{\eta\xi}\, e_{\eta\xi}\, dv \quad . \quad . \quad . \quad (10)$$

Vibration of structures having instability tendencies

$$\frac{2\pi^2\omega^2}{g}\sum_j m_j (w^2_2)_j + \sum_i P_i w_{2i} - \tfrac{1}{2}\int_v \widehat{\eta\xi}\, e_{\eta\xi}\, dv \quad (11)$$

Simple vibrations of elastic structures

$$\frac{2\pi^2\omega^2}{g}\sum_j m_j (w_2^2)_j - \tfrac{1}{2}\int_v \widehat{\eta\xi}\, e_{\eta\xi}\, dv \quad . \quad . \quad (12)$$

It is seen from these expressions that the expected simplifications of (11) in the case of zero loads P_i is the same as (12), and similarly the case of zero motion in (11), $\omega^2 = 0$, is covered by (10). Nevertheless it is convenient to keep the separate expressions.

ANALYTICAL SOLUTIONS

The analytical method will be exemplified by the problem of the vibrations of a strut, which requires expression (11) (Fig. 1).

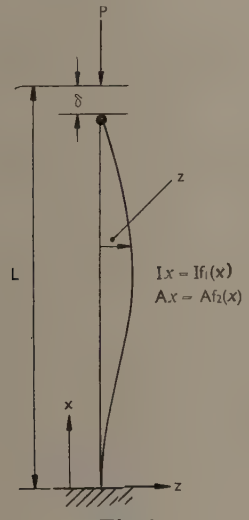

Fig 1

The strain energy is assumed to derive only from longitudinal stress and strain and may therefore be written

$$U = \tfrac{1}{2}\int_v \widehat{xx}\, e_{xx}\, dv = \frac{E}{2}\int_0^L I_x \left(\frac{d^2z}{dx^2}\right)^2 dx$$

The displacement w_{2i} is that of shortening due to the deformation $w_2 = z$ and thus the potential energy is

$$V = -\frac{P}{2}\int_0^L \left(\frac{dz}{dx}\right)^2 dx$$

The kinetic energy is given by

$$T = \frac{2\pi^2\omega^2\rho}{g}\int_0^L A_x z^2\, dx$$

where ρ is the density of the material of the strut and rotary inertia is ignored.

The quantity (11) to be minimized is therefore

$$H = \int_0^L \left\{\frac{2\pi^2\omega^2\rho}{g} A_x z^2 + \frac{P}{2}\left(\frac{dz}{dx}\right)^2 - \frac{E}{2} I_x \left(\frac{d^2z}{dx^2}\right)\right\} dx \quad (13)$$

The minimizing of such an integral is the problem of the calculus of variations. A number of texts are available on this subject, e.g. Fox.[5] Writing the integral as

$$H = \int_{x_1}^{x_2} F(x, z, z', z'')\, dx \quad . \quad . \quad . \quad . \quad (14)$$

The equation which z must satisfy to minimize H can be shown to be

$$\frac{\partial F}{\partial z} - \frac{d}{dx}\frac{\partial F}{\partial z'} + \frac{d^2}{dx^2}\frac{\partial F}{\partial z''} = 0 \quad . \quad . \quad . \quad (15)$$

Applying this to (13) yields the governing differential equation to the problem of the harmonic vibrations of a strut.

$$\frac{d^2}{dx^2}\left(I_x \frac{d^2z}{dx^2}\right) + \frac{P}{E}\frac{d^2z}{dx^2} - \frac{4\pi^2\omega^2\rho A}{gE} z = 0 \quad (16)$$

The solution of (16) will be of the form

$$z = c_1\,\psi_1(x, P, \omega^2) + c_2\,\psi_2(x, P, \omega^2) + c_3\,\psi_3(x, P, \omega^2) + c_4\,\psi_4(x, P, \omega^2)$$

and together with the four boundary equations will give four homogeneous linear algebraic equations in the arbitrary constants c_{1-4}

$$\begin{aligned} \alpha_{11} c_1 + \alpha_{21} c_2 + \alpha_{31} c_3 + \alpha_{41} c_4 &= 0 \\ \alpha_{12} c_1 + \alpha_{22} c_2 + \alpha_{32} c_3 + \alpha_{42} c_4 &= 0 \\ \alpha_{13} c_1 + \alpha_{23} c_2 + \alpha_{33} c_3 + \alpha_{43} c_4 &= 0 \\ \alpha_{14} c_1 + \alpha_{24} c_2 + \alpha_{34} c_3 + \alpha_{44} c_4 &= 0 \end{aligned} \quad . \quad (17)$$

For a non-trivial solution the determinant of the coefficients α_{ij} must be zero, i.e.

$$\Delta = \begin{vmatrix} \alpha_{11} & \alpha_{21} & \alpha_{31} & \alpha_{41} \\ \alpha_{12} & \alpha_{22} & \alpha_{32} & \alpha_{42} \\ \alpha_{13} & \alpha_{23} & \alpha_{33} & \alpha_{43} \\ \alpha_{14} & \alpha_{24} & \alpha_{34} & \alpha_{44} \end{vmatrix} = 0 \quad . \quad , \quad (18)$$

which gives an infinite sequence of characteristic values for ω^2. The lowest of these is the fundamental frequency of the problem. In the case in which the frequency is put equal to zero we obtain a sequence of solutions for P the lowest being the smallest buckling load. Re-substitution of ω^2 or P into (17) shows that one equation of (17) is linearly dependent upon the others so that only the ratio's $\frac{c_2}{c_1}$, $\frac{c_3}{c_1}$ and $\frac{c_4}{c_1}$ may be obtained. This is an expression of the well known result that the amplitude is indeterminate. Often the procedure outlined above is not possible, or at best, is very involved and requires the solution of a complex transcendental equation. In such cases it is often possible to employ one of the following approximate methods to obtain a solution.

THE RAYLEIGH-RITZ METHOD

For problems of this type Rayleigh[6] proposed that the form of deformation be approximated by a single function and Ritz[7] later generalized this to a finite series

$$z = a_1\varphi_1(x) + a_2\varphi_2(x) + \ldots a_i\varphi_i(x) + \ldots a_n\varphi_n(x) \quad (19)$$

in which φ_i $(i = 1, 2, \ldots n)$ are arbitrary functions restricted only in that they satisfy the boundary conditions of the problem and are linearly independent. The a_i are a set of parameters, the values of which are to be determined. The procedure amounts to the substitution of a problem with n degrees of freedom for a problem with an infinity of freedoms. The functions φ_i are assumed to be differentiable to the required order and therefore they may be substituted into the energy integrals, which require minimizing, and which, after integration, become merely quadratic functions of the parameters a_i. The energy integral (14) is stationary if its partial derivatives with respect to the a_i are equated to zero

$$\frac{\partial H}{\partial a_i} = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad (20)$$

The Rayleigh-Ritz procedure has thus substituted n simultaneous algebraic equations (20) for the one differential equation (15) of the problem.

In the case of stability or vibration problems the equations (20) will be homogeneous and the technique of solution will be the same as for the equations (17).

The ease with which the computation is performed and the degree of approximation achieved with a given number of terms will depend upon the choice of functions φ_i. Structural problems generally give a good idea of the physical processes and it should not be difficult to choose functions of the correct type. Obviously, geometrical symmetry in the problem should be carried through to the approximating functions. Considerable advantages will ensue with the choice of orthogonal functions. This is discussed under the heading ' Orthogonality.' The use of the Rayleigh-Ritz method applied to simple problems is shown in examples 1 and 2.

LAGRANGIAN MULTIPLIER METHOD

In certain cases it is possible to use an approximating sequence of functions such as (19) which satisfies all the requirements except that it does not satisfy a boundary condition term by term. If this is the case then a ' constraining ' relationship must be imposed upon the sequence (19) to ensure that the sequence as a whole satisfies the boundary condition. Such a constraining relationship will be in the form of an equation

$$\Gamma\ (a_1, a_2, \ldots a_i \ldots a_n) = 0 \quad . \quad . \quad (21)$$

and may arise due to its being necessary, for example, to provide a specified slope or deflexion at a given point.

Lagrange's method of simultaneously minimizing the energy integral and satisfying the constraining relationship (21) is to minimize

$$H - \lambda\Gamma$$

with respect to a_i, where λ is an, as yet, undetermined parameter (multiplier). This gives the n equation

$$\frac{\partial H}{\partial a_i} - \lambda\frac{\partial \Gamma}{\partial a_i} = 0 \quad . \quad . \quad . \quad . \quad . \quad (2$$

Taking the equations (21) and (22) there is a total $n + 1$ for determining the values of the $n + 1$ unknowns $a_1, a_2, \ldots a_n, \lambda$. A solution is therefore possible and in the case of stability or vibration problems it is obtained by equating to zero the determinant of the coefficients of a_{1-n} and λ from (21) and (2 which is now of order $n + 1$.

In the case of there being more than one constraini relationship (say there are m)

$$\begin{aligned} &\Gamma_1\ (a_1, a_2 \ldots a_n) = 0 \\ &\cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \\ &\cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad (2 \\ &\cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \\ &\Gamma_m\ (a_1, a_2 \ldots a_n) = 0 \end{aligned}$$

then, in the first instance, a function of the form

$$H - \lambda_1\Gamma_1 - \lambda_2\Gamma_2 - \ldots . \lambda_m\Gamma_m$$

will have to be minimized to obtain the n equations

$$\frac{\partial H}{\partial a_i} - \lambda_1\frac{\partial \Gamma_1}{\partial a_i} - \ldots . \lambda_m\frac{\partial \Gamma_m}{\partial a_i} = 0 \quad . \quad . \quad (2$$

These together with the constraining relationships (2 provide $n + m$ equations for the $n + m$ unknow $a_1, a_2 \ldots a_n, \lambda_1, \lambda_2, \ldots \lambda_m$.

It is found that where the number of degrees freedom is restricted to $n < \infty$ the values obtained frequencies and critical loads will be greater than t exact values. If the sequence (19) is extended to infinite series then the problem returns to one with infinite number of degrees of freedom and the resu obtained will usually be the exact result, althou Fox[5], p. 180, states that there is no certainty that th will always be the case. However, the process is n generally possible since it will involve the evaluati of an infinite determinant in most cases.

In certain cases it is possible to obtain an estima of the accuracy of the approximate value since t Lagrangian multiplier method may be used to obta a lower bound or limit to the estimated value. T procedure is to release some of the constraints b placing some of the a_i equal to zero in the constrainin equations $\Gamma_j = 0$. The result will be satisfactory pr vided the original sequence included sufficient terms obtain a close upper limit in the normal approximatio It is not appropriate to go into further detail here b reference may be made to the paper of Budiansky.[2]

GALERKIN'S METHOD

An alternative to the Rayleigh-Ritz method has bee proposed by Galerkin. A solution of the form (19) assumed but instead of substituting this into the energ integral (14) as in the Rayleigh-Ritz case the n equ tions of the form

$$\int_{x_1}^{x_2} \varphi_i \left\{ \frac{\partial F}{\partial z} - \frac{d}{dx}\frac{\partial F}{\partial z'} + \frac{d^2}{dx^2}\frac{\partial F}{\partial z''} \right\} dx = 0 \quad (2$$

...e used for the determination of the a_i and the deform-...ion, frequency or buckling load. The function in ...renthesis is the $L\,H\,S$ of the governing differential ...quation (15) of the problem into which has been ...bstituted the assumed solution (19). The n equations ...5) derive from successively putting $i = 1, 2, 3 \ldots n$.

In the case of the vibrating strut previously discussed ...e Galerkin equations (25) will be

$$\int_0^L \varphi_i \left\{ \frac{d^2}{dx^2}\left(I_x \frac{d^2z}{dx^2} \right) + \frac{P}{E}\frac{d^2z}{dx^2} - \frac{4\pi^2\omega^2\rho A_x}{gE} z \right\} dx = 0 \qquad (26)$$

...herein, as before

$$z = \sum_{i=1}^{n} a_i \varphi_i$$

In the case of deflexions of a simple beam the Galerkin ...quations are

$$\int_0^L \varphi_i \left\{ EI \frac{d^4z}{dx^4} - q(x) \right\} dx = 0 \quad . \quad . \quad (27)$$

...where $q(x)$ is the loading function.

The justification for this procedure has been dis-...ussed by Duncan[3], and whilst a general proof of its ...equivalence to the Rayleigh-Ritz method is laborious ...the demonstration in the case of a beam, equation (27), ...is straightforward.

The Galerkin procedure may also be used when the differential equation for the deflexion is in a form other than that of (15) which was given by the variation of the energy integral (14).

For example, if in the case of the beam the second order bending equation $EI \frac{d^2z}{dx^2} - M(x) = 0$ is used instead of the fourth order loading equation then the multiplier φ_i representing a deflexion must be replaced by the multiplier $\varphi_i'' = \frac{d^2\varphi_i}{dx^2}$ representing a change in rotation. The Galerkin equations for a beam may therefore be written in the alternative form

$$\int_0^L \varphi_i'' \left\{ EI \frac{d^2z}{dx^2} - M(x) \right\} dx = 0$$

where $\varphi_i'' = \frac{d^2\varphi_i}{dx^2}$ and $M(x)$ is the moment function.

ORTHOGONALITY

In all three methods mentioned above the substitution of the sequence of assumed functions into the energy integral will result in integrals of the form

$$\int_0^L \varphi_i\, \varphi_k\, dx\,; \quad \int_0^L \varphi_i'\, \varphi_k'\, dx\,; \quad \int_0^L \varphi_i''\, \varphi_k''\, dx$$

and the evaluation will be considerably simplified if the integrals are zero when $i \neq k$ (note : i and k must be integers). Functions which have this property are termed orthogonal functions and their use is preferred when possible. The most familiar orthogonal functions are the harmonics thus :

$$\int_0^L \sin\frac{i\pi x}{L} \sin\frac{k\pi x}{L}\, dx = \begin{cases} 0 \text{ for } i \neq k \\ \frac{L}{2} \text{ for } i = k \end{cases}$$

$$\int_0^L \cos\frac{i\pi x}{L} \cos\frac{k\pi x}{L}\, dx = \begin{cases} 0 \text{ for } i \neq k \\ \frac{L}{2} \text{ for } i = k \end{cases}$$

and it is because of their orthogonal properties that Fourier series are used so much in mechanical problems.

There is a considerable number of functions which have this orthogonal property and it is useful to note that the analytical solutions to vibration or stability problems yield orthogonal functions in their sequences of modes of vibration or buckling and therefore it is often possible to use the solutions for simpler problems as a convenient starting point for an approximate solution of more complex problems, particularly with the Lagrangian multiplier approach.

Bleich[8] notes that if orthogonal functions are applied to the solution of a stability or vibration problem by the Lagrangian multiplier method in which the constraining relations (21) are linear functions of the parameters a_i then the equations (23) express the undetermined multipliers directly in terms of the a_i and the order of the determinate to be solved is reduced from $m + n$ to m.

Example No. 1

The Stability of a Testing Machine

A number of testing machines have been known to exhibit instability by lateral displacement of the machine head when compression tests are being carried out on pin ended struts. The following is an analysis of that problem and shows the interesting result that there is a critical ratio of length of specimen to length of machine for complete stability above which tensile tests may cause instability and below which compression tests will cause instability.

Consider the idealized machine as indicated in Fig. 2.

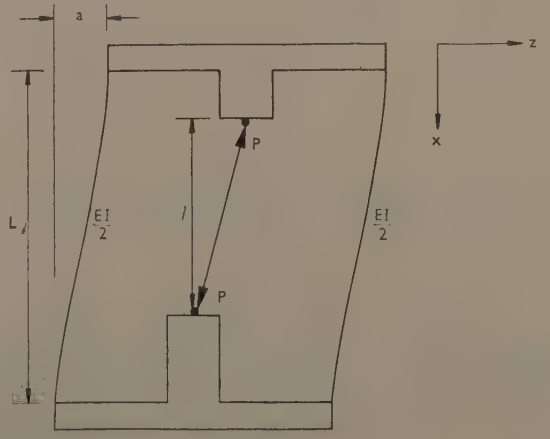

Fig 2

Since interest is directed towards static instability equation (11) is to be used with the frequency equated to zero.

The function to be minimized is

$$-(U+V)$$

Assuming I, the sum of the second moment of area of the columns of the machine, to be constant

$$U=\frac{EI}{2}\int_0^L\left(\frac{d^2z}{dx^2}\right)dx$$

The work done on the load by the structure consists of two terms :

$$V_1=\frac{P}{2}\int_0^L\left(\frac{dz}{dx}\right)^2dx$$

due to a movement against the direction of the load induced by the bending and

$$V_2=-P\triangle$$

where $\triangle$ is the change in length of the specimen due to its lateral displacement, and is approximately $\frac{a^2}{2l}$

Thus the expression to be minimized is :

$$H=-\frac{P}{2}\int_0^L\left(\frac{dz}{dx}\right)^2+\frac{Pa^2}{2l}-\frac{EI}{2}\int_0^L\left(\frac{d^2x}{dx^2}\right)^2dx$$

Assuming for z a function involving only one parameter

$$z=\frac{a}{2}\left(1+\cos\frac{\pi x}{L}\right)$$

Then

$$H=-\frac{Pa^2\pi^2}{16L}+\frac{Pa^2}{2l^4}-\frac{EIa^2\pi^4}{16L^3}$$

$$=a^2\left\{\frac{P}{2}\left(\frac{1}{l}-\frac{\pi^2}{8L}\right)-\frac{EI\pi^4}{16L^3}\right\}$$

Differentiation is trivial in this case and the energy expression may be equated directly to zero, giving

$$P=\frac{EI\frac{\pi^4}{L^3}}{\left\{\frac{8}{l}-\frac{\pi^2}{L}\right\}}$$

If the denominator of this expression is zero

$$l=\frac{8L}{\pi^2}$$

gives the specimen length for complete stability.

If $l<\frac{8L}{\pi^2}$ the critical load is positive giving instability for compression tests.

If $l>\frac{8L}{\pi^2}$, P becomes negative and the possibility exists of instability when the machine is performing a tensile test.

Example No. 2

The Frequency of a Tall Self-Supporting Chimney

This problem and a solution were given by the auth by Mr. Scruton.[9]

In a number of instances it has been found that t chimneys are caused to vibrate by winds of modera velocity. The wind passing the cylindrical chimn produces vortices which break away from alterna sides at regular periods and cause an oscillatory fo to be exerted on the chimney at right angles to t direction of the wind. The period of this force cy occurs in the time taken for the wind to advan through a distance of 4.5 times the chimney diameter Thus in order to determine the aerodynamic stabili of such a chimney it is necessary to find if the natu frequency of the chimney will coincide with the f quency of the vortex forces for the expected range wind velocities.

The natural frequency of such a tall chimney will influenced considerably by the potential energy var tions due to self weight during a deformation from t vertical position, and indeed, as will be seen, is intimate connected with the rate of self loading which wou cause general instability.

In order to generalize the problem somewhat it is be assumed that the chimney is of constant avera diameter but that the wall thickness decreases linear with height.

Thus the area is given by, approximately,

$$A_x=2\pi Rt_0\,(1-kx)$$

and the second moment of area by

$$I_x=\pi R^3t_0\,(1-kx)$$

The three energies may now be written, Fig. 3.

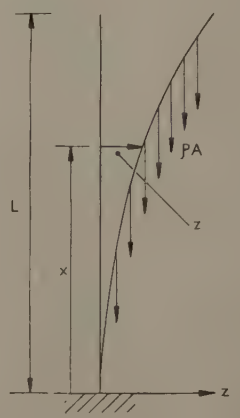

Fig 3

$$U=\frac{E}{2}\int_0^L I_x\left(\frac{d^2z}{dx^2}\right)^2dx$$

$$V=-\frac{\rho}{2}\int_0^L\left(\frac{dz}{dx}\right)^2\left\{\int_x^L A_x dx\right\}dx$$

and

$$T=\frac{2\pi^2\omega^2\rho}{g}\int_0^L A_x z^2\,dx$$

it is assumed that the mode of vibration is of the form

$$z = a\left(1 - \cos\frac{\pi x}{2L}\right)$$

hen the Hamiltonian action integral becomes

$$I = \int_0^L \left\{ \frac{E\pi^5 R^3 t_0 a^2}{32L^4}(1-kx)\cos^2\frac{\pi x}{2L} - \frac{\rho\pi^3 R t_0 a^2}{4L^2}\left[\int_0^L (1-kx)\,dx\right]\sin^2\frac{\pi x}{2L} - \frac{4\pi^3\omega^2\rho R t_0 a^2}{g}(1-kx)\left(1-\cos\frac{\pi x^2}{2L}\right)\right\} dx$$

nd the expression for the frequency becomes

$$\omega^2 = \frac{E\pi^2 R^2 g}{128\rho L^4}\left(\frac{0.5 - 0.149kL}{0.226 - 1.11kL}\right)\left[1 - \frac{8\rho L^3}{E\pi^2 R^2}\left(\frac{0.149 - 0.116kL}{0.5 - 0.149kL}\right)\right] \quad . \quad (1)$$

Considering the case of no motion it is possible to obtain n expression for the density ρ_{crit} which will cause nstability, which is

$$\rho_{crit} = \frac{E\pi^2 R^2}{8L^3}\left(\frac{0.5 - 0.149kL}{0.149 - 0.116kL}\right) \quad . \quad (2)$$

whilst if the potential energy term is made zero this results in the standard expression for a cantilever frequency

$$\omega_1^2 = \frac{E\pi^2 R^2 g}{128\rho L^4}\left(\frac{0.5 - 0.149kL}{0.226 - 1.11kL}\right) \quad . \quad (3)$$

The results (2) and (3) can be substituted into (1) giving

$$\omega^2 = \omega_1^2\left(1 - \frac{\rho}{\rho_{crit}}\right) \quad . \quad . \quad . \quad . \quad (4)$$

If the problem is simplified to one of constant section $I = I_{x=0}$ throughout the length then

$$\omega^2 = \frac{0.34EIg}{\rho AL^4}\left[1 - \frac{\rho AL^3}{8.3EI}\right]$$

the critical density becomes

$$\rho_{crit} = \frac{8.3EI}{AL^3}$$

and the " cantilever beam " frequency becomes

$$\omega^2{}_1 = 0.34\frac{EIg}{\rho AL^4}$$

whilst the result (4) still applies.

It is to be noted that in the case of a constant section the chosen form is not the most suitable and gives results which are 7 per cent. too high in the case of the square of the frequency and 6 per cent. too high in the case of the critical density. A better choice in this particular case is the deflexion curve under distributed load of a simple cantilever, viz. :

$$z = a\left(\tfrac{1}{2}L^2x^2 - \tfrac{1}{3}Lx^3 + \frac{1}{12}x^4\right)$$

The interesting result of equation (4) has been noted by Lurie[11] who has made it the basis for the estimation of critical loads. He notes that equation (4) is analytically true for only a limited range of structural conditions, although the difference is in general quite small.

Example No. 3

Deflexion of a Square Plate Supported on its Four Corners and Loaded Uniformly over its Area

The problem of a plate supported only on its corners does not appear to have been solved by classical methods but an approximate solution may be obtained by energy methods (Fig. 4).

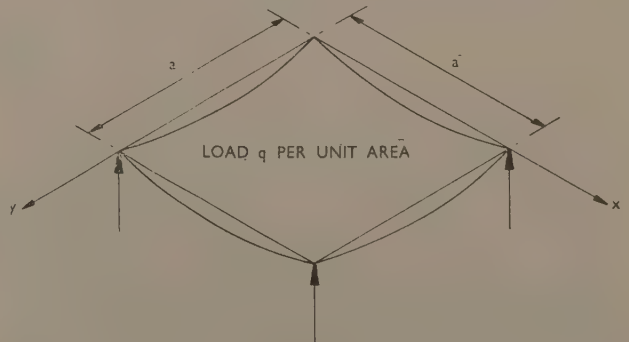

Fig 4

The governing differential equation of a flat plate of constant thickness is

$$\frac{\partial^4 w}{\partial x^4} + \frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} - \frac{q}{D} = 0 \quad . \quad . \quad . \quad (1)$$

where w and q are functions of the co-ordinates. In this particular case it is assumed that the loading q is

constant over the whole slab and the deformation will be taken to be of the form

$$w = \sum_{j}^{n} Aj \left(\sin \frac{j\pi x}{a} + \sin \frac{j\pi y}{a} \right) \quad . \quad . \quad . \quad (2)$$

This satisfies the deformation boundary conditions but does not give an entirely stress-free edge.

The Galerkin equations for the determination of the A_j are

$$\int_0^a \int_0^a \left(\sin \frac{j\pi x}{a} + \sin \frac{j\pi y}{a} \right) \left\{ \frac{\partial^4 w}{\partial x^4} + \frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} - \frac{q}{D} \right\} dxdy = 0 \quad . \quad (3)$$

Inserting expression (2) for w into equation (3) gives

$$\int_0^a \int_0^a \left(\sin \frac{j\pi x}{a} + \sin \frac{j\pi y}{a} \right) \left\{ \frac{\pi^4}{a^4} \sum_{i=1}^{n} i^4 A_i \left(\sin \frac{i\pi x}{a} + \sin \frac{i\pi y}{a} \right) - \frac{q}{D} \right\} dxdy = 0$$

which gives on integration

$$\frac{\pi^4}{a^2} A_j j^4 + \frac{8\pi^2}{a^2 j} \sum_{i=1,3}^{n} A_i i^3 = \frac{4a^2}{j\pi} \frac{q}{D}$$

This gives a general expression

$$Aj = \frac{4a^4 q}{j^5 \pi^3 D \left[\pi^2 + 8 \sum_{i=1,3}^{n} \frac{1}{i^2} \right]} \quad . \quad . \quad . \quad (4)$$

If (2) is made an infinite series then A_j may be written

$$Aj = \frac{2a^4 q}{j^5 \pi^5 D}$$

and the central deflexion of the slab is

$$w_{\text{central}} = 2 \sum_{j}^{\infty} Aj (-1)^{\frac{n-1}{2}}$$

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Acknowledgments

The author wishes to thank Mr. Tottenham for his hel comments and suggestions and Dr. Robinson for his assist in the preparation of the examples. The work described in paper was carried out at the Research Station of the Cement Concrete Association and is published with the permission of Director.

Book Reviews

La Fabrication et l'Utilisation des Tuyaux en Béton Armé et non Armé by A. Guerrin and G. Daniel. (The Manufacture and Utilization of Concrete and Ferro-Concrete Pipes, by A. Guerrin and G. Daniel). (Editions Eyrolles, Paris.) 130 pp. 6½ in. × 10 in.

There are no doubt a good number of Civil Engineers and Contractors who have had to find the answer to difficult problems when ordering concrete or ferro-concrete pipes. This book will give them the necessary data, since it gives complete and useful information concerning the manufacture of various types of pipes : compressed, vibrated and spun pipes.

An interesting chapter is the one dealing with the tests on pipes in liaison with the calculations and conclusions which can be drawn from these correlati Also worth mentioning is the chapter dealing w the practical methods of laying the pipes and questions dealing with the porosity of the pipes, watertightness of the joints, corrosion, etc.

The Authors conclude with valuable information the costs and specific advantages of ferro-concr pipes.

This book completes the volume which the s authors published a few months ago on the calculati of concrete and ferro-concrete pipes.

P. J. C

(*continued on page 194*)

The Bearing Capacity of Eccentrically Loaded Foundations on Sandy Soils

By W. Eastwood, B.Eng., Ph.D., A.M.I.C.E., A.M.I.Struct.E.

Introduction

IN the past it has been usual when designing eccentrically loaded footings to assume that the pressure distribution across the base of the footing would be linear and that the maximum value of the pressure must be limited to that permissible under a centrally loaded footing.

Recently, however, Meyerhof[1] has suggested that a more valid approach is to treat the eccentrically loaded foundation as if it were centrally loaded but had a width equal to the actual width less twice the eccentricity (see fig. 1). The results of experiments on small scale model footings are reported which appear to confirm this hypothesis. For footings on sand the Meyerhof theory gives somewhat smaller ultimate loads than the older theory except at small eccentricities.

At the time that paper was published the author was investigating the effects of eccentricity on strip footings on sand. His experiments on larger scale models than Meyerhof's gave results which are not in agreement with that author's findings, and the object of this paper is to report the results of these experiments and to postulate a possible reason for the disagreement.

The author's tests have also indicated that there may not be an ultimate load in the accepted sense if the footing is completely restrained from slipping sideways. The load increases indefinitely with increasing penetration of the footing.

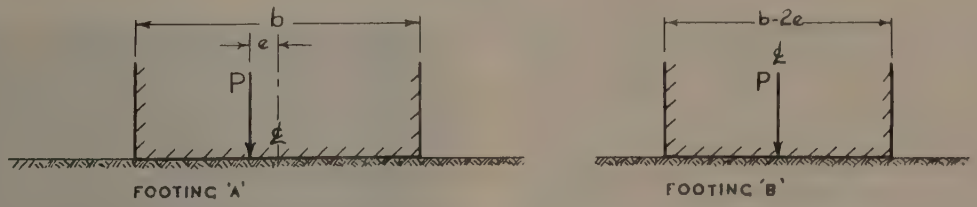


Fig. 1. Eccentrically loaded Footing 'A' assumed to be equivalent to centrally loaded Footing 'B'

DETAILS OF APPARATUS AND INVESTIGATION

All the tests were carried out on sand in a timber box braced with steel. The size of the box was 10 ft. x 5 ft. x 3 ft. deep.

The footings tested were cut from rolled steel channel and were either 6, 8 or 10 in. wide. Grooves in which the knife-edge loading device was located were cut at various eccentricities (see Fig. 2). The thickness of the channel under the knife edge was so small that the eccentricity was not appreciably affected by the tilt of the channel under the eccentric test load. The eccentricities used were up to b/6 (i.e. loading within the middle-third) except for the 6 in. wide footing. In this case the maximum eccentricity was almost b/4, i.e. loading well outside the middle-third.

Fig. 2 also shows the method of applying the load. To ensure that the footing could tilt endways or sideways without any restraint from the loading arrangement, the various pinned joints were incorporated. The length AB was also made fairly large (12 in.) so that, if the footing moved laterally relative to the loading beam during a test the inclination of the thrust through the loading arrangement would change only a negligible amount from the vertical. It was found in the tests that at the time the ultimate load was reached this change of inclination was never greater than about $\frac{1}{2}°$, and generally was much less.

It had been noticed in an earlier investigation that even with a centrally applied load there was often a measurable lateral movement before the ultimate load was reached. With eccentric loading it was thought that this lateral movement might be considerably increased unless some restraint were applied. In an actual structure there may be little or no restraint against lateral movement as in the case of a footing at the bottom of a relatively slender column, or the structure may be so stiffened that lateral movement would be insignificant. Accordingly, it was decided to repeat all the tests using two separate loading arrangements, one with lateral restraint of the footing and one without (see Fig. 2).

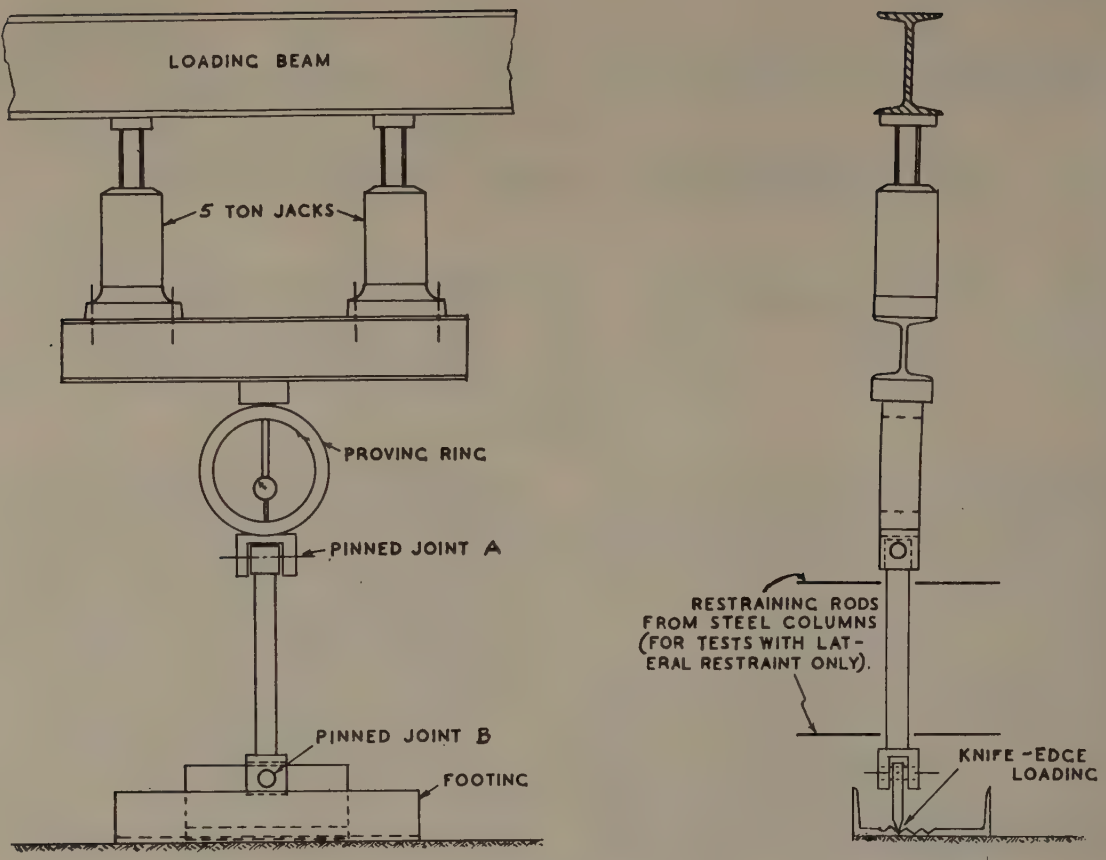


Fig. 2. Loading arrangement

The sand used in the tests had a grading curve as shown in Fig. 4. It was placed in 9 in. layers and compacted to refusal by a 'Kango' hammer. The average density obtained was 108 lb. per cu. ft. After each test the sand was dug over to a depth of 18 in. (i.e. greater than the measured depth of disturbance of the sand in a test) and recompacted. The compaction was continued until a straight screed run along the top edges of the box produced negligible scrapings. As the weight of sand in the box was unchanged the mean density of the sand was also constant from one test to another.

The lateral slip of the footing was measured continuously during the tests by means of micrometer dial gauges. A 30 in. long pointer, part of which can be seen attached to the far end of the footing in Fig. 3, moving over a stationary scale gave the angle of tilt of the footing, a correction being applied to the scale reading to take account of the vertical settlement.

During a test the footings were driven into the sand at approximately 1 in. per minute, measured at the point of application of the load.

RESULTS OF TESTS

(a) Mode of Failure

(i) Footings not restrained against lateral movement

In the tests in which the footings were not restrained against lateral movement the load increased steadily with settlement until slip surfaces were suddenly developed. This development of slip surfaces was quite audible and the load instantly dropped to about half its ultimate value. When the footing was driven still further into the sand the load slowly increased again, but in general the ultimate load had not been redeveloped at several times the settlements at which it was first attained.

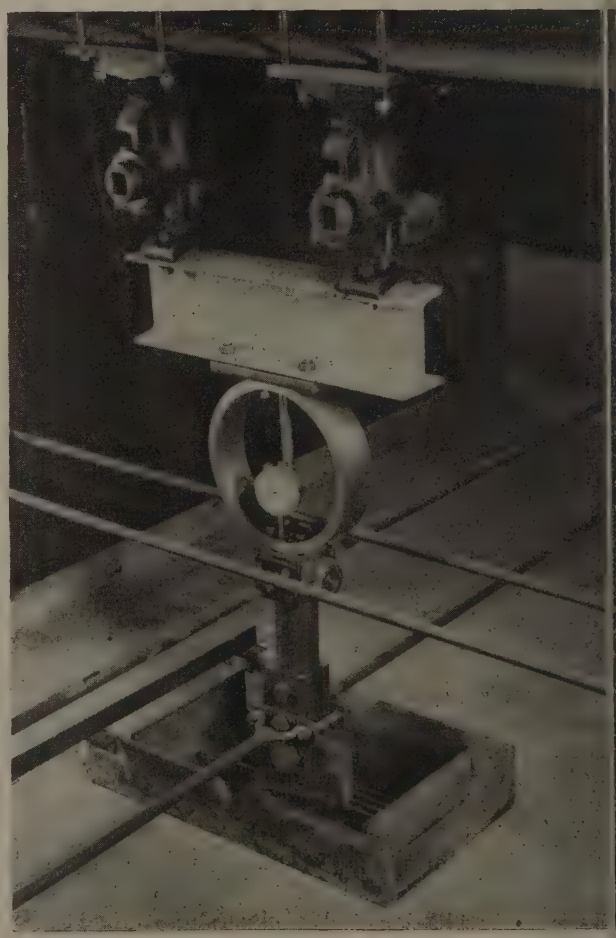


Fig. 3. Photograph of test on footing restrained against lateral slip

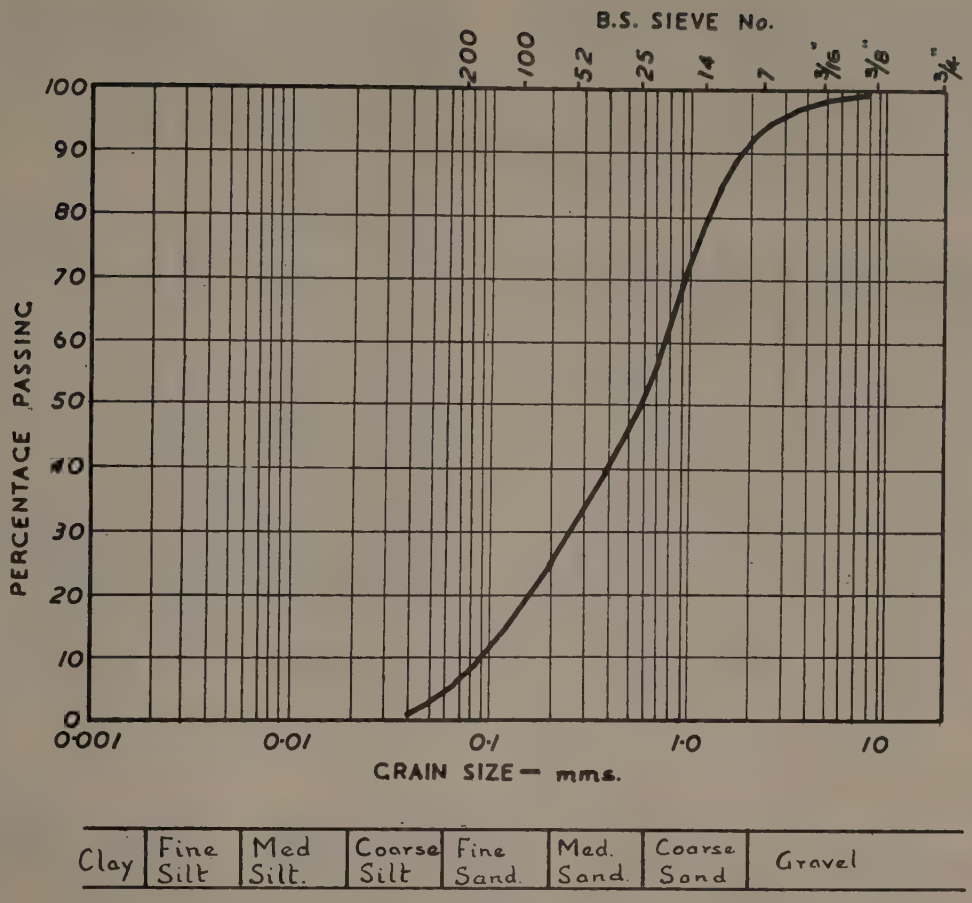
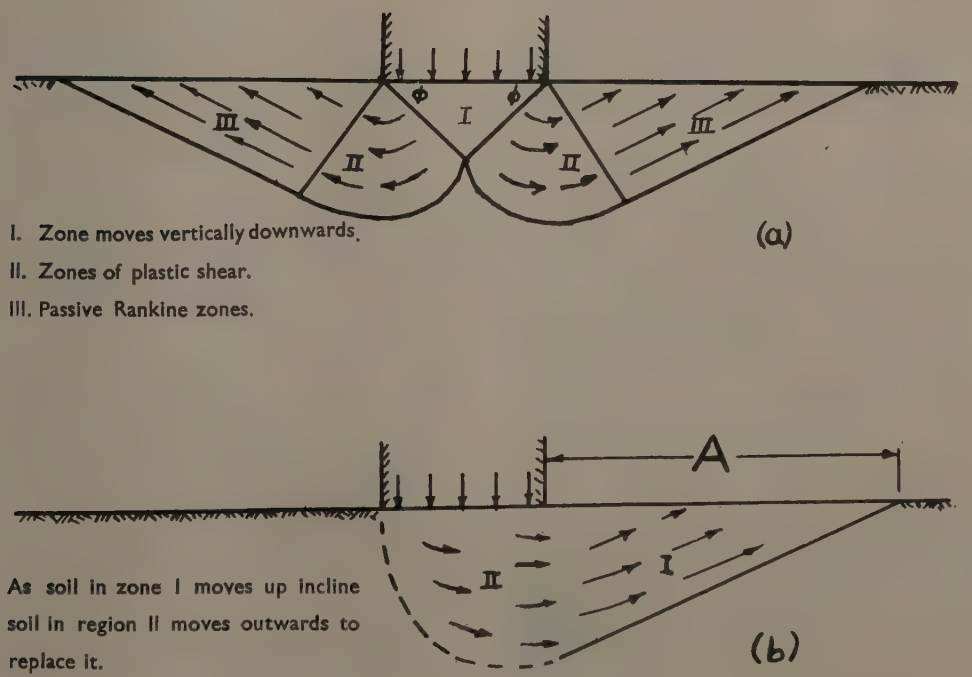


Fig. 4. Grading curve of sand used in tests



I. Zone moves vertically downwards.

II. Zones of plastic shear.

III. Passive Rankine zones.

As soil in zone I moves up incline soil in region II moves outwards to replace it.

Fig. 5. Comparison of two-way failure assumed in some theories and one-way failure which occurs in practice

Several of the more notable theories of foundations are based on the assumption that for centrally loaded foundations failure occurs by sliding wedges being formed on both sides of the footing (see Fig. 5a). Most investigators have found, however, that in practice failure occurs by sliding to one side only (see Fig. 5b). This was so in every test except one of the present series, the dimension A in Fig. 5b being about 4b on the average for central loading.

Although only one slip surface was formed at the ultimate load, a further surface did develop later if the footing was pushed far enough into the sand. With central loading this second sliding surface was usually on the same side of the footing as the first, but in about 25 per cent. of the tests it was on the opposite side. With eccentric loading the first sliding surface was invariably on the same side as the eccentricity (see Fig. 6), and if the footing was then pushed further into the sand a second surface was usually formed further out on the same side. The ' outcrops ' of these first and second surfaces were roughly at distances of b and up to 4b from the footing respectively, this latter dimension tending to be less for tests with large eccentricity.

In at least one of the well-known foundation theories it is assumed that the footing fails by rotation about some centre as shown in Fig. 7a. The tilt measurements showed that the rotation was always away fro the slip surface as in Fig. 7b. Generally the angle tilt when the ultimate load was reached was abo 1° for centrally loaded footings, and as much as for eccentrically loaded footing, increasing with t eccentricity.

(ii) Footings restrained against lateral movement

There were some important differences in behavio when the footings were restrained laterally.

With central loading on the 8 in. and 10 in. wi footings the first sliding surface to form sometim outcropped at approximately one footing width aw on the side towards which the footing tilted, and som times at about three footing widths out on the opposi side. There was not, however, a sudden drop bearing power when this first surface formed as is i variably the case with unrestrained footings. T load continued to rise as the footing was driven furth into the sand, although at a decreased rate in gener Even when further sliding surfaces were formed the was no drop in the load, and it appears that there no clearly defined ultimate load in the usual sense.

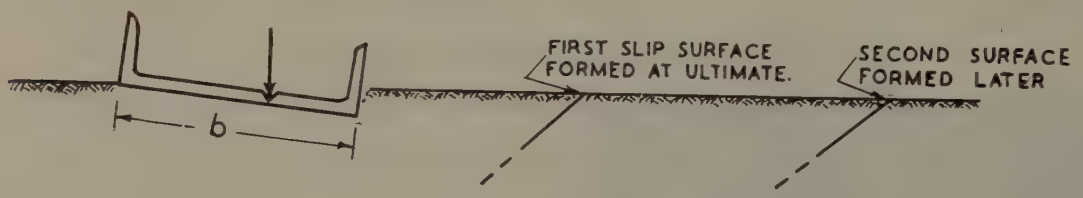


Fig. 6. Position of slip surfaces in tests

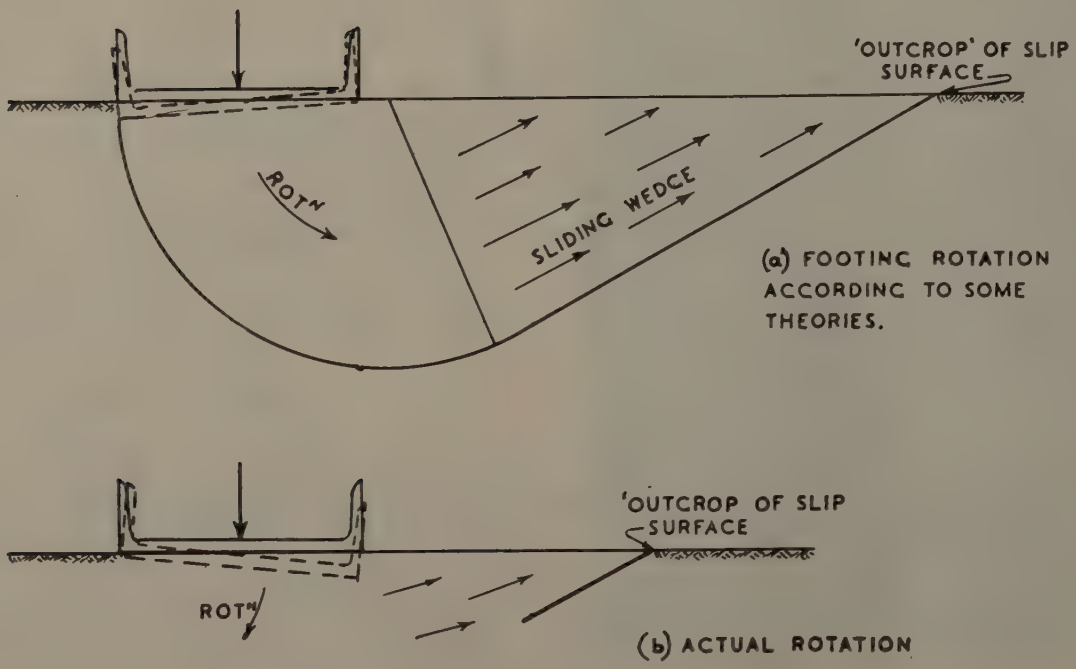


Fig. 7. Direction of rotation of footings according to some theories and actual direction as observed in tests

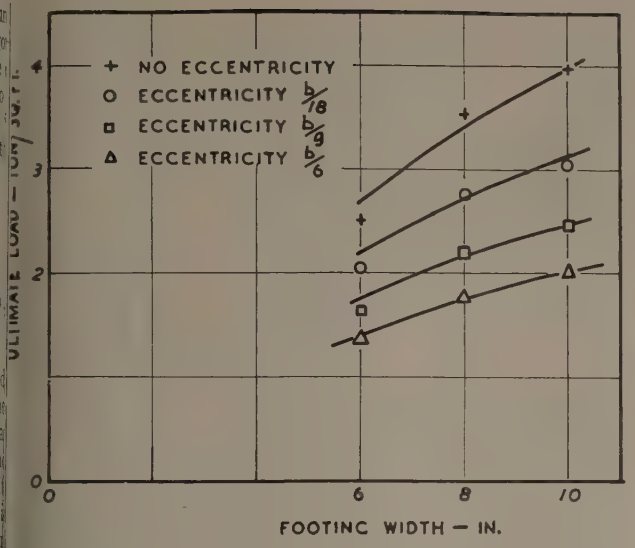


Fig. 8. Variation of ultimate load with footing width and eccentricity of application (no lateral restraint)

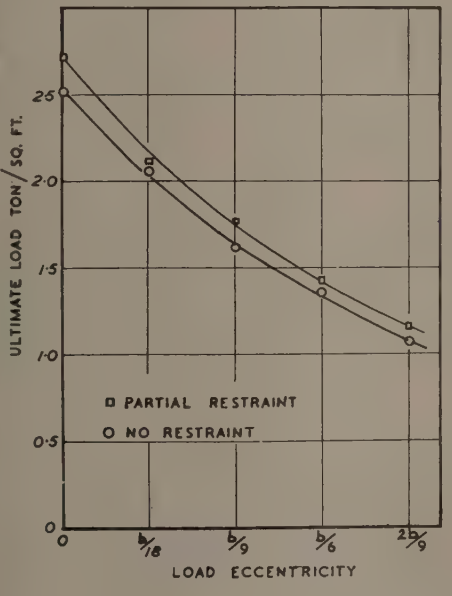


Fig. 9. Effect of partially restraining lateral movement on ultimate load for 6 in. wide footing

Similar behaviour was recorded for eccentric loading on the 8 in. and 10 in. wide footings.

Unfortunately, the restraining device was incapable of preventing lateral movement completely because of the slight play which was necessary to allow unrestricted vertical movement. The magnitude of this lateral movement varied between 0.01 and 0.025 in. This is quite small compared with lateral movement in unrestrained tests (generally between 0.1 and 0.2 in.) but was sufficiently large nevertheless for an ultimate load to be obtained in the normal way for some of the 6 in. footing tests. The value of the ultimate load obtained in these tests was somewhat larger than in the completely unrestrained tests as will be seen later.

(b) Ultimate loads

The average ultimate load for three tests with each set of conditions is given in table 1.

Load eccentricity	18in x 6in footing		18in x 8in footing		18in x 10in footing	
	No lateral restraint	Laterally restrained	No lateral restraint	Laterally restrained	No lateral restraint	Laterally restrained
zero	2.52	2.73	3.53	No definite Ultimate	3.98	No definite Ultimate
b/18	2.06	2.12	2.72		3.04	
b/9	1.63	1.77	2.20		2.46	
b/6	1.36	1.43	1.78		2.01	
2b/9	1.08	1.16				

Table 1.

Average ultimate loads in ton per sq. ft. for tests on sand with various eccentricities of loading.

The results for the tests without restraint are plotted in Fig. 8. It will be seen that the results for all eccentricities are in accordance with previous investigations using central loading, the load per unit area for a given eccentricity ratio being not quite proportional to the footing width.

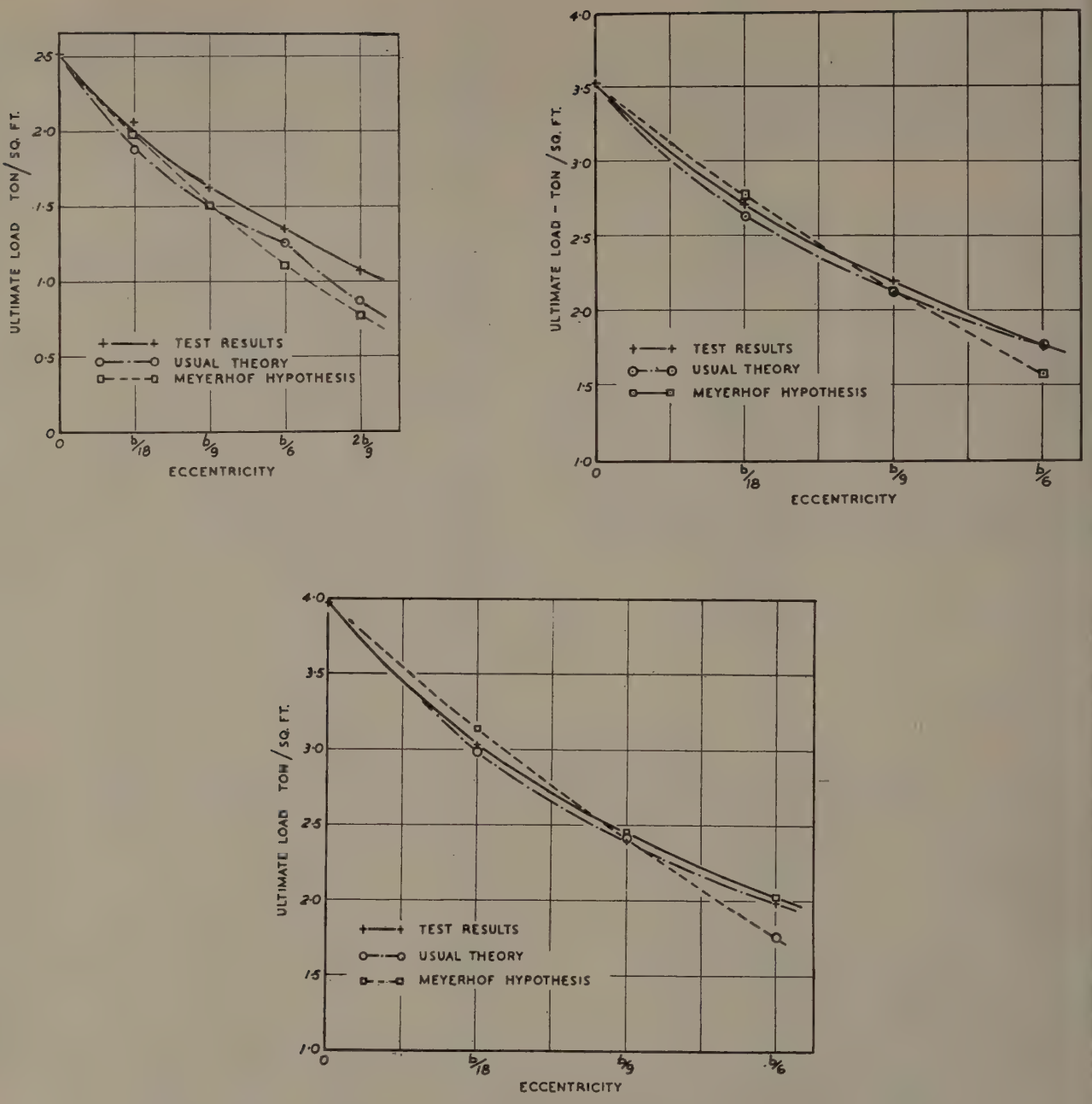
The results for the tests with and without lateral restraint for the 6 in. wide footings are compared in Fig. 9. From this figure it will be seen that the partial restraint caused an average increase in the ultimate load of about 6 per cent.

Figs. 10 a, b and c compare the variation of bearing value with eccentricity which was obtained in the tests with that which would be expected according to the Meyerhof theory, and also with the older theory in which a straight line pressure distribution, the maximum value of which is constant, is assumed. It will be seen that for the 8 in. and 10 in. wide footings the older theory gives bearing values in closer agreement with the experiments than the Meyerhof theory throughout the range of eccentricities. With the 6 in. footing each of the theories agrees more nearly with the experiments over certain ranges, but the older theory has a smaller maximum deviation and also has the advantage that it does not give an over-estimate for any eccentricity.

DISCUSSION OF RESULTS

The present tests are obviously insufficient in scope to draw general conclusions for all types of footing. But they do indicate that under certain conditions at least the old-established assumptions may be better than Meyerhof's suggested alternatives. Since the tests were very similar to some of those carried out by Meyerhof except that they were to a much larger scale (Meyerhof's footings were only 1 in. wide in general) a reason was sought for the apparent differences in the two sets of results.

A probable reason for the divergence is apparent from an examination of Meyerhof's apparatus and method of test. Photographs in his paper show that the loads were applied by an arrangement approximately as in Fig. 11. It will be seen that as the footing tilted under the action of eccentric load the eccentricity would also increase. As no measurements of the angle of tilt appear to have been made it is probable that no correction was made to the eccentricity value. Thus the ultimate loads obtained will correspond to greater eccentricities than the reported values. The



Figs. 10a, b, and c. Comparison of test results with usual theory and Meyerhof hypothesis for 6in., 8in., and 10in. wide footings

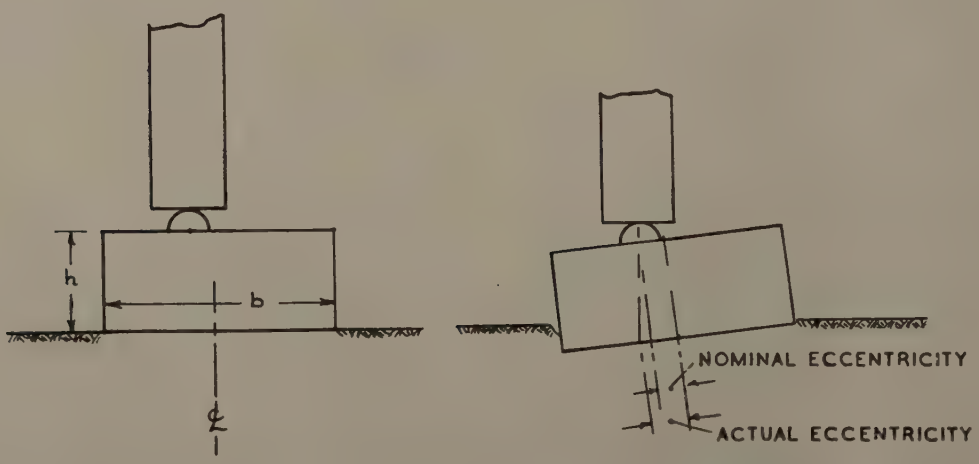


Fig. 11. Change of eccentricity as footing tilted in Meyerhof's tests

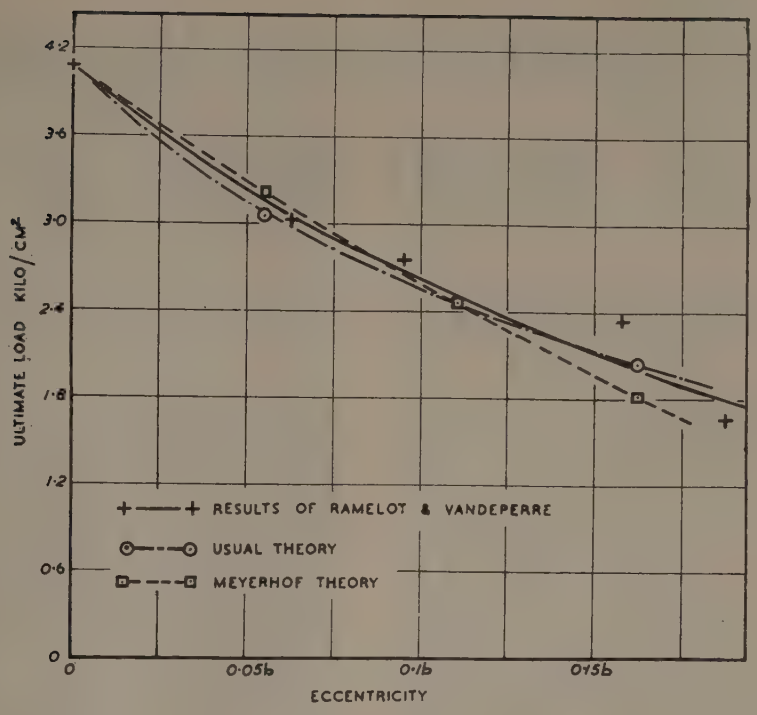


Fig. 12. Comparison of Ramelot and Vandeperre's test results on square footings with the usual theory and the Meyerhof hypothesis

error in the eccentricity was probably quite considerable if the angle of tilt was as great in Meyerhof's tests as it was in the author's. At ultimate load the value varied between about 0.5° with central loading to about 6° when the eccentricity was b/6. Thus if the ratio h/b in Fig. 11 were say 0.5, the actual eccentricity at ultimate load in a test in which it was initially b/6 would be b/6 + b/20 approximately.

Ramelot and Vandeperre have also reported the results of some tests on square and circular footings on sand, using eccentric loading. The author has compared their results for square footings with the usual theory and the Meyerhof hypothesis. Although footings of various widths were tested, only in the case of the 30cm. square footing were sufficient repeat tests made to draw an accurate graph of ultimate load against eccentricity for eccentricities up to b/6. This graph is shown in Fig. 12. There is some scatter of the points partly because some of them represent only a single test, but again it will be seen that the usual theory is upheld rather than the Meyerhof hypothesis.

CONCLUSIONS

(1) The tests carried out by the author on strip footings indicate that the usual practice of assuming that there is a straight line distribution of pressure under an eccentrically loaded foundation, and that the ultimate value of that pressure is the same as that under a centrally loaded foundation, is sound for footings on sand and eccentricities up to b/6. An alternative hypothesis put forward by Meyerhof, in which the ultimate load for an eccentrically loaded footing is assumed to be equal to that for a footing of width equal to the actual width minus twice the eccentricity, does not agree so well with some of the experiments.

(2) Footings on sand which are restrained from slipping sideways have no definite ultimate load whether centrally or eccentrically loaded, there being no sudden drop of bearing power when slip surfaces are formed. When only partial restraint against lateral movement is allowed, there may be a well-defined ultimate load somewhat higher than that obtained with no lateral restraint of the footings.

Acknowledgments

The experimental work described in this paper was carried out in the University of Aberdeen, with the active help and encouragement of Professor Jack Allen, D.Sc., M.I.C.E.

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Fabrication and Erection of Steel Plate Girder Railway Bridges

Discussion on the Paper by Mr. M. F. Palmer, M.I.C.E., M.I.Struct.E.

Discussion

Mr. L. E. Hawkins said he had been concerned with two of the bridges and he was very interested to read in Mr. Palmer's Paper about the Southern Railway work.

Referring to the London Transport bridges, Mr. Hawkins said the bridge at New Cross, curiously enough, had one outside and two " centre " girders ; one of the girders, outside in the picture, was the same size as the centre girder and had been provided as a centre girder for an additional track which was never put in. A particularly weak feature of the old bridge was, of course, the lighter outside girder. The other two girders, although corroded, were not so bad but the floors were very bad and it had not been possible to provide new floors and preserve the old wrought iron girders. The head-room over the canal was a difficulty, as the railway had to rise from an overbridge a little way away to get over the canal. The new bridge was so designed that the underside of the cross girders was exactly flush with the underside of the main girders. The cross girder connections were set up on to the main girders in order to save every bit of head room.

An interesting point about the Galena Road bridge was that the Act of Parliament under which the bridge had been constructed required a span of something like 50 ft. although the road was a little cul-de-sac leading to a few cottages and was not occupying anything like the full span. Again there was very great difficulty with construction depth because the bridge was just at the top of the steep rise from the tunnel at Hammersmith to the viaduct at Ravenscourt Park. The track could not be raised and it was very difficult job to design a bridge for the full span and headroom. It seemed a great waste to provide a bridge of that span to cross such a little road so an endeavour had been made to get the span reduced. However, the most the authorities would concede was two lines of columns which, if ever the time came to widen the road, would come on to the edge of pavements ; as the road is now restored, it does not occupy the full width between the columns.

The reduced span between columns had made it possible to use a deck span. There were no main girders but joists running between the columns with a lower span at each side over what might at some later date be footpaths. It had been impossible to adopt the usual arrangement of rolling in, as mentioned in the Pap because there were two bridges each carrying two trac i.e. four tracks altogether—those for the District a Piccadilly railways—and so the bridges were c structed from underneath and lifted up by jacks.

It appeared that the Southern Railway bridges h steel plate floors and a waterproof layer ; if Mess Joseph Westwood & Co. Ltd., were responsible for waterproofing, possibly Mr. Palmer could give so particulars about it and say whether that waterpr layer on steel plates, with the deflection of plates un the load, was entirely satisfactory. His own organi tion's general practice was to concrete in floors so and although that did mean a great deal of extra de load on the bridge, from his own calculations, even w the little extra steel required in the main girders carry the dead load of the concrete, it was appreciall cheaper than a steel deck floor consisting of continuo steel plates.

Referring to the New Cross bridge, which was ove canal, Mr. Hawkins said tenders had been invited p ticularly from firms that it was thought could load st on to a ship or a barge. He had always thought th transport by water was the cheapest means of transp and it had occurred to him that the main girders co be most cheaply brought to the site by water, b apparently none of the contractors who tendered h considered the possibility. It had been anticipat that the main girders would be transported in t pieces and spliced on the site, but they were made one piece. They were over 90 ft. long and were one the longest and heaviest loads moved in London. Th went by road from Millwall to Marylebone, where th were loaded on to rail and then went by devious rou round to New Cross, on the Southern Region, wh they got on to his own system.

Mr. Stanley Vaughan (Vice-President), referring the Stewarts Road bridge, a section of which was sho on page 323, said the bottom flange was in two separ parts and each part seemed to be unduly eccentric o wards in relation to the web and top flange. T would result in considerable secondary stresses and presumed that these secondary stresses had be allowed for in the design. No doubt there had be some very good reason for adopting this particu design notwithstanding the consequential seconda stresses, and it would be interesting to hear what th reason was.

Mr. J. A. Williams (Member), referring to the pai ing of the steelwork, asked (a) was the painting specif by the engineer or was the contractor invited to subm proposals ; (b) what in fact was the painting specifi tion adopted, and (c) what was the pre-treatment of steelwork?

* *Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on the 16th December, 1954. Dr. S. B. Hamilton, M.Sc., B.Sc.(Eng.), M.I.C.E. (President) in the Chair. Published in " The Structural Engineer," Vol. XXXII, No. 12, pp. 322-332. Dec. 1954.*

Mr. S. M. Reisser (Member) asked whether Mr. lmer could say something about comparative costs th similar bridges of a riveted construction.

Mr. B. L. Clark (Associate-Member) said the one all thing which intrigued him was the curvature on e Twickenham Fly-over bridge, and he wondered how le stresses were dealt with in that : did Messrs. West-ood do the design for these girders and were the condary stresses actually gone into, or were they in ct done by some other method ?

Referring to the use of the low hydrogen welding rod, r. Clark asked whether it had been found in the thick nges that it was really worth using that rod instead the normal or cheaper grade of welding rod ?

Mr. P. L. Capper (Member of Council), referring to e Stewarts Road bridge, said a question had already en asked about the apparent eccentricity of the ttom flange, but as he read the drawing the top nge was also split at the centre so that the two girders longing to adjoining spans could act separately, ex-pt for the cover plates which connected them together. the design, what distribution of load was allowed for om the girder supporting one track to the next ?

Mr. H. D. Osborn (Associate-Member), referring to e Stewarts Road bridge (Fig. 3) and the Twickenham y-over bridge (Fig. 10) said the photograph showed e girder assembly jigs and he wondered whether con-deration had been given to using cradles to carry the gs to construct the girders in, in order to turn them er and thus speed construction.

Fig. 12 showed a very heavy load for the Twickenham y-over being carried by road, and it would be interest-g to have the author's comments on that in view of e fact that it had recently been remarked in the Press at legislation might be introduced in order to restrict avy loads on roads throughout the country, and con-equently construction of that sort might be greatly estricted in the future.

Mr. M. F. Palmer, replying to Mr. Hawkins' question ith regard to the asphalt on the steel-plated deck, said is firm had certainly heard of no trouble in that respect. Actually he believed a number of tests had been arried out recently, some of them by the Road Re-search people, and his information was that the laying f asphalt coatings on steel decks was quite satisfactory. At the present time his firm was working on another ridge and they intended to use that method of con-truction for the decking. It was true that the steel eflected, but the asphalt would follow that deflection ithout trouble.

With regard to Mr. Hawkins' question as to the trans-ort of those girders which were 95 ft. long and weighed bout 35 tons, by water to the site, rather than by road nd rail, the difficulty was getting them on barges. The rdinary dock-side cranes were for handling much maller pieces than that and his firm had not a wharf rane capable of handling 35 tons. Splicing large girders at site was in itself a costly operation.

Mr. Vaughan had referred to the bottom flanges being eccentric to the webs, and although Messrs. Westwood had not designed the bridge Mr. Palmer's view was that the reason for it was to give access to the underside of the girders for maintenance. He felt sure that maintenance had been one of the principal things in mind when the bridge was designed : very little of it showed above the track so it was not very difficult to maintain the top, and the underside could be got at very easily. Naturally, the welded construction was also a great help in so far as maintenance was concerned, the surface being smooth as against the presence of rivet heads in riveted construction.

The units were really a series of channels, and Mr. Capper had asked what precautions had been taken regarding the different deflections of adjacent spans. His firm had not been responsible for the design of that bridge but it would appear that the two halves had been designed as a single girder and that there were sufficient connections between the two half girders to transmit the load.

When they tendered for the job Messrs. Westwood had visualised that having fixed one span and ballasted it, the next span when placed would have no ballast on it and therefore would not deflect the same amount which would give trouble in making the connections between those girders, but in fact they had started bolting up from either end and there had been no trouble at all in matching the holes.

He did not know whether everybody had appreciated the point he had been trying to make in the Paper, but his firm had been desirous to avoid putting small holes in the stiffeners to take the cover plates and then having to reamer them out with the two halves of the girders together, because the distance between the webs was only about 16 ins. What had been done was to drill full size the covers and the stiffeners before they were welded into the girder. They were welded in, bolted together and after completion the girders were broken apart. No trouble was experienced and when the spans were brought together later the holes matched. That idea might be carried further and it might be possible to build quite a large structure in the works with turned bolted joints already in some of the members, and the whole structure welded up. Then one could break the turned bolted joints, transport the job to the site, and bolt it up again just as it was in the shops.

Mr. Williams had raised the question of painting, but there had been no special painting on any of the contracts. They were all well wire brushed and had an initial coat of red lead, followed by the railway specification of two or three coats of further paint, although he was not too sure of the type of paint. Possibly Mr. Hawkins would be able to say something about the London Transport bridges in that connection.

Mr. Reisser had asked for comparative costs between the welded bridges and the riveted ones, but such a comparison was difficult to make. There had certainly not been two designs to tender on, and Stewarts Road bridge was in itself a prototype. It could no doubt have been built by riveting but it would have been awkward. So far as the comparative costs of riveted and welded girders generally were concerned his feeling was that if the welded girder was designed properly there was very little in the final cost.

On the question of low hydrogen rods there were probably some electrode experts present and he himself would like to hear what they had to say, but he had not considered that there was any alternative to low hydrogen rods for welding the thick flanges to the webs. It had been suggested that ordinary rods might have been used, and if by " ordinary " rods were meant the usual mild steel rods then he believed there would have been trouble with cracking. As was mentioned in the Paper, contact electrodes were used for welding the stiffeners, and actually although they were a fairly expensive electrode they had saved a lot of time and no doubt in the end they were cheaper and they gave a very good job. With those electrodes the stiffeners were welded

on the girder when it was in the sloping position, and also it had been possible to weld down from the top, whereas with an ordinary vertical weld they would have welded up from the bottom.

Mr. Osborn had mentioned the possibility of putting the welding jigs in cradles but his firm had been concerned about the twisting which might occur in a girder and rather favoured having fixed jigs so that the girder was rigidly held. When one had overhead cranes in the shop capable of lifting a girder in its entirety it did not take long to put chains round it and turn it over. They had used circular cradles in the past but they had not been satisfactory.

It had seemed certain that the question of large loads would crop up in view of the recent mention in the Press, but in fact traffic had not been interrupted very much. The Police were in sole charge of the loads through London and the Stewarts Road spans which were about 50 ft. long and 11 ft. or 12 ft. wide were taken through London on Saturday afternoons and he had been surprised to see them going along at about 15 miles an hour and giving no trouble at all. The Twickenham girders had to be held outside the works until night-time and went through London between about eleven o'clock at night and three o'clock in the morning, again without any trouble. On the other hand he sympathised with the person who had raised the question because last year he had been in a string of motorists two or three miles long following a big barge which was being moved by road.

The subject of brittle fracture was touched on in the Paper and he believed there was to be a separate Paper on that later in the Session, but it had proved to be a very controversial question.

Mr. Hawkins, referring to the question of the paint, said the New Cross bridge was mostly riveted, although some of the parts were welded before they were assembled. The webs were made up of several pieces welded and the cross-girder ends were specially shaped by welding. There were two coats of red lead on contact surfaces before riveting, but no other painting until the whole bridge was erected, giving a chance for weathering of mill scale. After wire brushing and chipping, there was one coat of red lead as a rust inhibitor, and two coats of silica-graphite, then one finishing coat of a proprietary paint of brown colour, adopted because it was the same colour as stain which gets on all London Transport bridges from break-block and rail wear dust.

Mr. G. B. Raper (Member), referring to the bridges which were filled in with concrete, asked whether it was the usual practice to paint the steel before concreting or to concrete against the steel.

Mr. Palmer, in reply, said it was not customary to paint the steel when it was to be concreted but it was, of course, well wire brushed.

Mr. B. L. Clark said presumably Mr. Palmer had avoided the question on curvature because he had not designed the bridge, but he had asked whether the full calculations had been gone into.

Mr. Palmer replied that he was unable to say much about the design because his firm had not been responsible for it, but the Stewarts Road bridge was very carefully tested under several conditions of loading and in all cases the stresses were found to be under the maximum permissible stresses and generally in accordance with the design stresses.

Mr. Bullen pointed out that the Stewarts Road bridge was not in fact the curved one.

Mr. Palmer, in reply, said he had missed the point but although the bridge had not been designed by his firm, he was sure full calculations would have been made. This Fly-over bridge at Twickenham had not been tested.

Mr. Reisser, referring to the question as to whether it was correct to use low hydrogen electrodes on thick flanges, said he could only express the personal opinion that low hydrogen electrodes gave less trouble. On the other hand there were thousands of girders with very thick flanges which had been welded with ordinary electrodes long before anybody ever heard of low hydrogen electrodes. They were perfectly satisfactory and it was just a question of temperature, the conditions under which one was going to weld, and the precautions one took, and, of course, a question of economics because low hydrogen electrodes cost more than the ordinary ones.

Mr. H. C. Gibson said that he had been concerned with the erection of the Stewarts Road bridge and although Mr. Palmer was correct in saying that they had not been bothered with deflection they had been concerned about its effect on lining up the holes and had kept a 100 ton jack on site in case the holes would not come together. The difficulty had been that the loaded half-girder had leaned towards the track a little so that when the new half-girder had been brought alongside the two sections were a little open at the top and it had been necessary to put clamps across the top to hold the sections together while they were bolted up. In his view the stiffening joist should have been a little longer to prevent this " leaning ".

Mr. M. F. Palmer, in reply, said probably what had happened was something similar to what had been experienced in the works, when it had been necessary to force out the top flanges while welding the deck because the contraction of the welds at the ends of the cross-girders tended to bring in the tops of the girders. The point he had made in the Paper was in connection with the relative deflections of the girders, whereas the point raised by Mr. Gibson must, he felt, be due to the welding of the cross-girder ends.

In constructing the curved bridge, when the web was welded centrally to the flanges the curved flanges tended to straighten out. It would seem the curving of the flanges introduced internal stresses and that when the welding was done the heat released some of them enabling the girder to slightly straighten out. There had in fact been a loss of curvature of about $1\frac{1}{2}$ in. in 60 ft.

The Chairman thanked Mr. Palmer for having dealt with the questions, and the proceedings then terminated.

Correspondence

The Institution, whilst being at all times pleased to open its columns to correspondence, cannot accept responsibility for the opinions expressed.

o the Editor of THE STRUCTURAL ENGINEER

Draft Revision to B.S.449

Sir—In view of the importance of British Standard pecifications to the structural engineering profession, erhaps you could publish in THE STRUCTURAL ENGINEER he following remarks regarding the draft revision to .S.449 which has been circulated to obtain comments om the industry.

I feel that the membership of the Institution would e doing a good service to the profession if their comnents, possibly as a result of this letter, were placed efore the responsible British Standards Committee.

Firstly, although my comments are critical, there are ood points in this draft and improvements to the .S.449-1948, but with these I shall not concern myself ere. Some of these criticisms, and the list is by no neans exhaustive, are equally applicable to the present .S.449-1948.

There are various clauses which are spoilt by vagueess, e.g. in Clause 11c(ii), could not examples be given f types of cladding, coming under the heading of ormal permeability? Again Clause 42a, dealing with leductions for holes, states " For countersunk rivets or olts an appropriate addition shall be made to the liameter of the hole." Why not be more definite? Another, Clause 60a, states " Members in bracing systems shall be of rigid section," and might just as well be omitted for all the assistance it affords. Other Clauses are equally vague.

At least one Clause (No. 57) seems to me to state the obvious, and yet another, viz :—

> " *Note.* In built up members where the chains of holes considered in individual parts do not coincide with a common chain of holes in the whole member, the effect of any bolts or rivets joining the parts between such non-coincident chains of holes shall be taken into account in determining the strength of the member."

This seems meaningless, though doubtless it was inserted for some good reason.

Another fault is typified by Clause 43a which says " . . . shall be reduced in the ratio of . . ." instead of the simpler and clearer . . . shall be multiplied by . . .

On perusal of other clauses one cannot help but feel that engineers are having a specification thrust upon them by people who are fortunate enough not to have to carry out designs in accordance with it. Bearing in mind the fact that in a practical structure many of the applied loads are estimated (which probably means guessed intelligently) and cannot be obtained otherwise than by empirical rules, e.g. wind pressure, superimposed loading on floors, etc. etc., is it really worth the effort and is it not a waste of the designer's time, especially as he is usually trying valiantly to satisfy an all too early tender date, to go through the mental gymnastics required to comply with Clauses 22b(i) Table 18, 22b(ii), 43a and 43c?

Surely a straight line formula such as

$$F_{bc} = \frac{1{,}000}{\frac{l}{r_{yy}}} \times K_1$$

or the older

$$F_b = 14.4 - 0.22\frac{l}{b}$$

is to be preferred from practical considerations to that of the new Clause 22b

$$F_b = \frac{100{,}000}{(l/r_y)^2}\sqrt{1 + \frac{1}{20}\left(\frac{l}{r_y}\cdot\frac{t_e}{d}\right)^2} + K_2\frac{100{,}000}{(l/r_y)^2}$$

for checking bending stresses in beams suffering from lateral instability in view of the large number of beams to be dealt with on most contracts and the fact that the first mentioned formula can be reduced to a coefficient which simply requires to be divided by the span l. The new formula is too cumbersome to be dealt with in this way and after all, the latest handbooks, which have not long been published, deal neatly with the problem in accordance with the present B.S.449. No doubt there are good reasons for the new formula, but one must suppose that the same holds for the present one. I am, I hope, in favour of progress, but not at the expense of over-involved academic calculations founded on guessed loads. Life is altogether too short when it comes to checking instability in beams with curtailed flanges.

The design of single angle tension members seems unduly complicated by the clause stating that the effective area of the outstanding unconnected leg shall be reduced in the ratio of

$$\frac{1}{1 + 0.35\frac{b}{a}}$$

where b = area of outstanding leg, a = net area of connected leg. It seems to me that the present method is complicated enough and has the advantage that tables are already prepared and published to simplify the work. The question is this : " Is the change really justifiable? " It is only too easy to make alterations to the existing specifications, undoubtedly altering the supposed stresses in the various members of a structure by trifling amounts, but leaving the scantling to be used exactly the same. Is this progress?

The following practices, standard since 1948, seem to me somewhat illogical :

(*a*) Limiting the bending stress in a welded plate girder to 9.5 tons/sq. in. whilst allowing 10 tons/sq. in. in the case of a riveted compound beam ;

(*b*) Limiting the allowable $\frac{l}{r}$ of one component of a battened or laced strut to the arbitrary figure of 50 in spite of the fact that the $\frac{l}{r}$ of the compound strut may be 180, 250 or even 350.;

(*c*) The decision (see Figs. 16 and 17, Appendix E) to assume that a slab base on a column is equivalent to a pinned end. Surely if under the most adverse loading conditions bearing exists over the whole cross

sectional area of the shaft, the condition is that of a 'flat end' or 'fixed end.' I do know of cases prior to 1948 where this was assumed successfully in the design of multi-storey office blocks and blocks of flats.

I think this draft should be very carefully revised in view of its great influence on British engineers and their designs, which may have to compete in overseas markets. One danger is that too much hack work checking stresses allowable by involved formulae may obscure the broader and more fundamental concepts of the structural design.

Finally, after so many destructive suggestions, may I presume to make a constructive one. Engineers are used to obtaining information from drawings and in any specification one thumb nail sketch would be wo a welter of words.

I seem to remember that it required at least text-book and several B.C.S.A. pamphlets to exp the intentions of the present B.S.449 despite its p iixity, and the new draft is even wordier.

May I commend to those responsible for the n draft revision the advice attributed to James Wa 'Of all things, but proverbially so in mechanics, supreme excellence is simplicity."

Yours, etc.,
L. KRAUS
(Associate Member).

Book Reviews

(*continued*)

Theorie Der Constructionen, by Dr. Ing. Konrad Sattler. (Wilhelm Ernst & Sohn, Berlin, 1953.) 300 pp. + XII; $6\frac{3}{4}$ in. × $9\frac{1}{2}$ in. Price 43 DM.

Structural engineers are not infrequently segregated into two groups; those who build in steel on the one hand, and on the other those who specialise in concrete. This distinction is reflected in the offer of alternative problems or design studies in professional examinations and prize competitions, and it is sometimes even recognised in the constitution of technical committees. For although concrete is now so infrequently used plain, the steel in reinforced and prestressed concrete is not recognised as an equal partner but as a secondary if essential element, like the alloying constituents which modulate the basic mechanical properties of steel or aluminium.

By reason of this two-party classification, in which we do not recognise the existence of a centre party, with roots on each side of the fence, and comparable interests in both fields, we have no idiomatic rendering of Dr. Sattler's title. Literally translated it is "Theory of composite structures," and under it the author, who is a professor at Charlottenburg University, discusses the general theory of steel and concrete used in combination. The reason for this new designation is easily seen in some of the author's examples in which concrete slabs are integrated with structural steel girders. Ordinary reinforced concrete is included, though not elaborated in great detail. In this treatment, prestressed concrete becomes a particular form of composite construction, and the author's approach to it from first principles, rather than with reinforced concrete as the point of departure, seems to lead to a wider view of the potentialities of prestressing. For although this is not so much a handbook of design as a treatise on design analysis, the author's interests are essentially practical rather than academic.

Dr. Sattler sets himself first to define, in mathematical terms, the basic properties of his materials, taking account of the creep and shrinkage of the concrete. His starting point is the assumption of an empirical expression for the elastic modulus, in terms of age and cylinder strength, based on the work of Bach, Schüle, Graf and Ros. For the empirical representation of creep and shrinkage, he relies on the fundamental research of Dischinger and others, part of which has been adopted in the German specification DIN 4227. From this well established base the author proceeds to discuss the effect of creep a shrinkage on a number of different types of compos structures, statically determinate and indetermina After a chapter on temperate stresses, the last third the book is devoted to examples, which include re forced columns, beams and arches. The most intere ing applications, however, are the bridge designs which steel girders of plate-web or truss type compounded with concrete deck slabs which may pre-stressed before or after bonding with the st girders. One particularly noteworthy example is sketch design for a bridge of 1,100 ft. span, of elega elevation, with separate spans of 325, 650 and 325 approximately translated from metres. This brid which with but little modification could be used span the Thames at Westminster with only two riv piers, consists of a sectionally prestressed concre deck carried on steel trusses with a depth of only 13 at the crown.

This is a treatise which is well worth translation in English, but even in the original it should not overlooked by research workers nor by enginee interested in making the most efficient use of steel a concrete in combination.

E. H. B.

Design of Concrete Structures (5th Edition), by L. Urquhart, C. E. O'Rourke, and G. Winter. (New Yo and London: McGraw-Hill, 1954.) $9\frac{1}{2}$ in. × $6\frac{1}{2}$ i 508 plus ix pp. Price 60s.

This book has been completely rewritten in order take into consideration the many new developmen and changes in design codes, but retains the sa method of presentation and general arrangement before. It deals with the behaviour and performan of reinforced concrete and its theory of design, and al with the practical design of many types of reinforc concrete structures. The section on rigid frames h been completely revised and includes a new treatme of slope deflection and an extensive discussion of t moment distribution method of analysis, and a ne section has been added dealing with the ultimate plastic design method, and with prestressed concret This is a comprehensive book on reinforced concre design and will be particularly valuable to all studen of the subject.

(*continued on page 196*)

Institution Notices and Proceedings

PRESENTATION TO THE SECRETARY AND THE ASSISTANT SECRETARY

For long enough inquisitive minds have sought to penetrate the mysteries of Honorary Office; at long last their inquisitiveness is to be satisfied. For only an event of " major " proportions could stir the Honorary Editorial slumbers and call forth the luckless sleeper to his duty. During thirty-three years our Assistant Secretary has grown " grey " in his labours; for twenty-five years our Secretary has directed our affairs.

The Institution was founded in 1908. Mr. Gray joined us in 1922, that is 33 years ago, and Major Maitland in 1930, that is 25 years ago.

" Men may come and men may go but they go on for ever," or so it seems.

To mark the occasion, members of the Institution have contributed to a fund. At the Annual Dinner on Friday, March 25th, 1955, Major Maitland and Mr. Gray were presented with albums containing the signatures of all contributors. Each received a cheque and each has had his portrait prepared for hanging in the Institution Headquarters. Certainly, the event is unique, and the members of the Institution must surely congratulate themselves upon the unstinting service rendered to them over these many years by their two senior officers. Not only, however, do members congratulate themselves, but in these tokens they seek to express their deep appreciation of all that these two gentlemen have achieved on their behalf over these many years. Let us then do honour to these two, and let us thank them for erecting upon a sure foundation so comely a structure.

HONORARY EDITOR.

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1., on Thursday, 28th April, 1955, at 5.55 p.m., Dr. S. B. Hamilton, M.Sc., B.Sc.(Eng.), A.R.C.S., M.I.C.E., M.I.Struct.E. (President) in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated, below, should be referred to when consulting the Year Book for evidence of membership.

STUDENTS

AKINTOYE, Raphael Olaniyi, of London.
ASKEY, Reginald Charles, of London.
BALASUBRAMANIAN, Trichinopoly Neelamengam, B.Sc. Madras, of Madras, India.
BROCHWICZ-LEWINSKI, Andrzej Maria Boleslaw, of London.
GREGORY, Michael John Partington, of Timperley, Cheshire.
HILTON, George Geoffrey, of Scunthorpe, Lincs.
LEAVER, Frank, of Adlington, nr. Chorley, Lancs.
MENZIES, James Charles, of Wellington, New Zealand.
TUCKER, Malcolm Bailey, of Crediton, Devon.
WAUGH, John Beverley Marriott, of Bulawayo, Southern Rhodesia.

GRADUATES

BUILT, John William, B.Sc.(Eng.) London, of London.
BURKE, Thomas John, of Johannesburg, South Africa.
BUSH, Leslie Norman, of Liverpool.
CLIFFORD, Paul Anthony, B.Sc.(Eng.) London, of Isleworth, Middlesex.
DATTA, Mrinal Kanti, B.E.(Civil) Calcutta, of Calcutta, India.
DATTA, Shubhendra Kumar, B.Sc., B.E.(Civil) Calcutta, of London.
DIAS, Ponnahennedige Negesha Kamal, B.Sc.(Eng.) London, of Redcar, Yorks.
DRUMMOND, Brian Godfrey, B.A.(Cantab.), A.M.I.C.E., of Hassocks, Sussex.
DUNTON, David Anthony, B.Sc.(Eng.) London, D.I.C., of London.
HILL, Graham Charles, of Hounslow, Middlesex.
HINGORANI, Narain Vasiomal, of London.
JEE, Raymond, B.Sc.(Eng.) London, of Gillingham, Kent.
JONES, Dennis Sidney, of Vanderbijl Park, Transvaal, South Africa.
KLEYNHANS, Evert, B.Sc.(Eng.) Stellenbosch, of Cambridge.
LOWES, George Edward, of Middlesbrough, Yorks.
MCKELVEY, William Leslie, of Belfast, N. Ireland.
SPALL, Norman John, A.M.I.C.E., of Harlow New Town, Essex.
WARD, Melville Pickering, of North Shields, Northumberland.
WILLIAMS, Brian Salathiel, B.A.(Cantab.), of London.

MEMBERS

BRODERICK, Stanley Reeves, M.I.C.E., of London.
HAYES, William Ivor, M.I.Mun.E., of Chesterfield, Derbyshire.

TRANSFERS

Students to Graduates

BOOTH, Richard, of Belper, Derbyshire.
MISZEWSKI, Jan Stefan, of London.
NADARASA, Mailvaganam, of London.
WHITE, Bryan John, of London.
WHITE, Dennis Jack, of Kidderminster, Worcs.

Graduates to Associate-Members

CHRISTIE, Dennis, of Gezira, Trucial Coast, Persian Gulf.
MCCAULEY, John Brian, of Newton-le-Willows, Lancs.
THRELFALL, Robert Malcolm, B.Sc.(Tech.) Manchester, of Kuala Lumpur, Malaya.
WEDGE, Henry Robert, of London.
WILLIAMS, Robert Raymond, of Liverpool.

Associate-Members to Members

HOCKLEY, Charles Henry, of Kingston-upon-Thames, Surrey.

Member—Retired Member

HOLMES, William Clifford Walker, of Durban, South Africa.

OBITUARY

The Council regret to announce the deaths of John Medows THEOBALD, O.B.E. (Member) ; Edward Prentice MAWSON (Retired Member) ; and Douglas Willerton RICHARDSON (Associate-Member).

EXAMINATIONS—JULY, 1955

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on the 12th and 13th July, 1955 (Graduateship) and the 14th and 15th July (Associate-Membership).

JUNE MEETING

An Ordinary General Meeting of the Institution of Structural Engineers will be held at 11, Upper Belgrave Street, London, S.W.1. on Thursday, 23rd June, 1955, at 5.15 p.m. This meeting is for the election of members and is open to corporate members of the Institution only.

REPRESENTATION

The Council have made the following nominations of members to represent the Institution :

South-East London Technical College, Building Advisory Committee—Lt.-Colonel G. W. Kirkland (Hon. Curator).

Northern Area Building Advisory Sub-Committee of the Regional Council for Further Education for the South West—Mr. C. J. D. Boxall (Member) : re-appointed for three years.

DRURY MEDAL AWARD

The fifth competition for the above award will take place in 1955. The subject is the design of a mobile crane.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary : envelopes to be marked in the top left-hand corner, " Drury Medal Award."

The closing date for the competition is October 1st, 1955.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say, primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution, or failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate in the presence of a Justice of the Peace or a Commissioner for Oaths.

RESEARCH AWARDS

The Council have instituted a Research Prize Fun from which awards may be made annually to the auth or joint authors of papers describing original resear which they have carried out. Research awards m be made for papers read at Headquarters or in t Branches and published in the Journal, or for pape published in the Journal only without being read at open meeting.

The assessment for such awards will be ma annually, but awards will be made only to the co tributors of such papers as reach a standard judged the Literature Committee to be satisfactory.

Work submitted under this scheme must be origin and may include any of the following :

(a) investigations of an experimental or analytic character ;

(b) studies of historical or statistical records ;

(c) improvements in principles or methods of co struction ;

(d) research into methods of structural engineeri and building, the nature and use of plant and t organisation of engineering work ;

(e) any related or combined studies which a deemed by the Literature Committee to be of research character.

In cases where the research work described in t paper was not the work of one individual, the names all the collaborators should be given in the paper.

Awards may take any or all of the following forms A research medal ; a diploma ; a money prize.

Application for consideration for a research awa must be made to the Secretary of the Institution, ar in preparing papers for reproduction in the Journa authors must comply with the conditions laid down f all such contributions. Particulars of these conditio may be obtained from the Secretary.

In judging research papers, the following factors w be considered :

(a) the nature of the subject and its conclusions ;

(b) the value of the paper in advancing the scien and art of structural engineering ;

(c) the standard of preparation and orderly arrang ment of the subject-matter.

Research papers will also be eligible for adjudicatio for the Institution Medals if they comply with t Regulations governing those awards.

The closing date for the receipt of applications respect of papers published in the Journal betwee October, 1954, and September, 1955, is October 3r 1955.

MACLACHLAN LECTURE COMPETITION, 195

The closing date for the receipt of entries for the nex Maclachlan Lecture Competition is Friday, March 30t 1956. The general conditions of the competition are a follows :

1. The Institution of Structural Engineers sha institute a written lecture to be known as the Maclachla Lecture and to be held annually.

2. The subject of the Lecture may be on any aspe of Structural Engineering as long as in every secon year the subject shall be confined to steel structure

3. Entrance into the competition for the Lectu shall be confined to Graduates and Associate Membe of the Institution who are under the age of 32 years.

4. All papers entered for the competition shall be ubmitted to assessors to be appointed by the Council the Institution, and all such papers (including the rize-winning Lecture) shall be available for publication the Journal of the Institution at the discretion of he Council.

5. No paper submitted shall have been published or ead elsewhere.

6. The winner of the competition shall be required o present the Lecture to a meeting of the Institution t which he will be presented with the sum of £17 10s.

7. Should a competitor's paper be considered worthy f ranking second in merit he will receive a consolation ward of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no ecture submitted is considered to be of sufficient merit o warrant award, or for any other reason, the Institution shall transfer the above sums to the Research Fund of the Institution.

Particulars of the Competition for 1956

1. The Maclachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1956.

2. The subject of the Lecture shall be on any aspect of structural engineering.

3. The work should be submitted as the script of a lecture which the author, if successful in the competition, will deliver before an audience in the course of about one hour. The development of mathematical formulæ and detailed calculations should be avoided as far as possible in the text ; if they are essential they should be embodied in appendices. Photographs, drawings, graphs, etc., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.

4. Six copies of each Lecture should be submitted and should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the Institution is Friday, March 30th, 1956.

YEAR BOOK AND LIST OF MEMBERS

The Year Book and List of Members for 1955 will go to press in July, for publication in October, when a copy will be sent to all members.

Members are requested to inform the Secretary of any alterations in titles, degrees or addresses, which have not already been notified, by July 4th, in order that such amendments may be included in the new edition.

REPRINTS FROM " THE STRUCTURAL ENGINEER "

Authors who wish to obtain reprints of their papers published in " The Structural Engineer " should notify the Secretary of the Institution not later than the 15th of the month in which the paper is published.

COMMUNICATIONS TO MEMBERS

The Institution is experiencing difficulty in receiving replies to communications addressed to the undernoted members, and the Secretary would be glad to be notified of their present addresses as soon as possible :

Members

G. M. Anderson
Major R. T. Cook
J. Swindells
Capt. R. T. Thadani
R. S. Thompson
J. R. Vethavanam.
R. C. Vaughan.
F. B. Watson

Associate-Members

W. W. Calder
T. Hibling-Keiller
S. A. Kikkeri
G. A. Matchett
L. J. E. Savidge
H. Waye
M. McG. Wright

Graduates

B. T. P. Lewis
A. Money
M. J. J. Reffitt

Student

T. W. Smallfield

ADDITIONS TO THE LIBRARY

Baroin, J. B. *Air Entrainment in Concrete with some French Examples.* Soc. Ing. Civils de France, London, 1954.

Clark, M. E. & Others. *Inelastic Behaviour of Ductile Members under Dead Loading.* Univ. of Illinois, 1954.

Davidson, J. F. *The Dynamic Lateral Instability of Beams.* A.D.A., London, 1955.

Huntington, W. C., & Others. *The Distribution of Concentrated Loads by Laminated Timber Slabs.* Univ. of Illinois, 1954.

Lewis, W. A. *Further Studies in the Compaction of Soil and the Performance of Compaction Plant.* Road Research Technical Paper, No. 33. London, 1954.

National Bureau of Standards Applied Mathematics Series 43. *Tables of Sines and Cosines for Radian Arguments,* Washington, 1955.

Road Research Road Note No. 19. *The Design Thickness of Concrete Roads,* London, 1955.

Shand, P. Morton. *Building : The Evolution of an Industry,* London, 1954. Presented by Mr. N. P. Keep.

Timoshenko, S. & J. N. Goodier. *Theory of Elasticity,* New York and London, 1951. Presented by Mr. J. A. Cole.

LONDON GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : J. A. Pope, 53, Cranleigh Drive, Leigh-on-Sea, Essex.

BRANCH NOTICES

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Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., P.O. Box No. 3306, Johannesburg, South Africa.

During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone : 34-1111, Ext. 257.

Natal Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co. Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

Book Reviews

(*continued*)

Analysis of Deformation (Vols. I & II), by Keith H. Swainger, Ph.D. (London : Chapman & Hall, 1954.) 285 pp. & 365 pp., $8\frac{1}{2}$ in. $\times$ $5\frac{1}{2}$ in. Price 63s. and 70s.

Dr. Swainger has, for many years, been endeavouring to win acceptance for a generalised linear theory of strain in elasticity and plasticity ; this treatise embodies his analytical theory, with practical applications.

Swainger's argument is briefly this : a point O in unstrained material becomes O′ in strained material, so that OO′ is the modulus of a spatial displacement vector U having components C(wb) representing whole body motion (rotation and translation), and D, representing pure straining displacement. In classical elasticity, the pure strain symmetric tensor and rotational skew tensor are obtained by unique resolution of the strain tensor. Swainger prefers dyadic analysis, but the difference is unimportant. Unrestricted integration is permissible if the components of these tensors are infinitesimals, because superposition can then be applied, and the constant of integration is the rigid body motion. Now, Swainger claims that suitable strain compatibility conditions can be found to ensure integrability, no matter how large these components are, and hence a linear theory of finite strain can be formulated. According to everyone else such a procedure implies that Curl U = Constant, i.e. irrotational strain, and this would nullify, for example, St. Venant's theory of torsion and bending, because a shear centre, i.e. the point through which the line of application of the load must pass to neutralise the torsional effect, could not exist in Swainger's theory. But Swainger claims that St. Venant is at fault.

In spite of abundant verification by careful experiment, Swainger's theories have been subjected to vitriolitic attack, and during the past ten years the epithet " false " has been applied more frequently to the work of Swainger than to that of anyone else ; but Swainger will have none of this—he fights back with polemical vigour : Cauchy, St. Venant, Love, Filon, Southwell, Hill, in fact, all have gone astray (except Swainger) ; one and all are accused of ignorance and inability to understand. So the highly original theory developed in this treatise is either fundamentally wrong (as indeed it is if Swainger and everyone else are speaking the same technical language), or else it is the testament of an outstanding genius whose revolutionary ideas may be obscured in faulty mathematical analysis. Unfortunately, Swainger doesn't help to win sympathetic understanding because his presentation is needlessly involved and often obscure ; and it is safe to hazard that practising, even mathematically-minded engineers will lay aside these superbly produced but highly-priced volumes with a feeling of utter bewilderment which, it may be noted, and that without injustice to Swainger, has already been experienced by many who have tried to understand Swainger's work ever since he published his first paper.

F. A. G.

The Ultimate Strength of Prestressed Concrete Beams

By Professor Henry J. Cowan, M.Sc., Ph.D., A.M.I.Struct.E.

FAILURE of a prestressed concrete beam in bending* is caused initially either by yielding of the steel near the bottom face (Primary tension failure), or by crushing of the concrete near the top face (Primary compression failure). Primary tension failures occur in beams with low steel ratios, whereas primary compression failures are to be expected in heavily reinforced beams; the two types of failure occur simultaneously for a certain limiting steel ratio r'_0 which depends on the properties of the two materials and the bond between them.

In general beams should have a load factor of at least 2 against primary tension failure, which depends essentially on the ultimate strength of the steel. The quality of concrete cannot be controlled as carefully as that of steel, and its ultimate strength is therefore more variable. The load factor against primary compression failure should therefore be at least $2\frac{1}{2}$.†

It is sometimes possible that the load factor against cracking is higher than that against ultimate failure. The formation of cracks in the concrete would then be followed immediately by collapse of the beam. This type of failure is evidently undesirable, and it should be avoided by increasing the ratio of the cross-sectional area of the steel to that of the concrete bottom (tension) flange.

Deformation of Concrete and Steel at High Loads

Below the visco-elastic limit the strain of concrete is roughly proportional to the applied stress. Although the linear relationship of Hooke's Law is modified by inelastic deformation due to creep, the departure from proportionality becomes appreciable only after a period of time.‡ When the failure of a concrete beam due to excessive *live* loads is considered, the effect of creep can be neglected. Above the visco-elastic limit the apparent strain increases more rapidly due to the breakdown of the adhesion between the aggregate and the cement paste, and the consequent formation of minute cracks (Fig. 1). These inelastic strains due to microcrack formation soon become much greater than the elastic strains, and it is therefore possible to replace the stress-strain curve by a simplified diagram, shown dotted in Fig. 1.¶ This consists of an elastic phase, in which the concrete strain, e_{ce}, is proportional to the stress, f_c,

$$e_{ce} = f_c/E_c$$

where E_c is the modulus of elasticity of the concrete. The ultimate compressive strength, $\varnothing_{cu}$, is reached at the "yield" strain, e_{cy}, and thereafter the strain increases without further increase in stress. The concrete crushes when the ultimate compressive strain, e_{cu}, is reached.

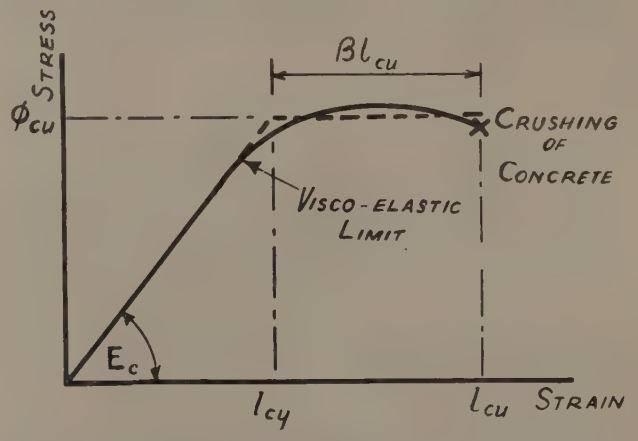


Fig. 1.—Actual and idealized stress-strain diagram for high strength concrete

There is, in fact, some decrease in concrete stress for high strains,§ but this can be allowed for by choosing a slightly lower value for $\varnothing_{cu}$. The ultimate compressive strength of the concrete in a prestressed or reinforced beam is generally assumed to be equal to the crushing strength of a prism or cylinder with a height/diameter ratio of 2.‖ Theories based on this assumption have shown mainly good agreement with experimental data. The cylinder crushing strength

* Bond and shear failures are not considered in this paper.

† "First Report on Prestressed Concrete," *Institution of Structural Engineers*, London, September 1951, p. 10.

‡ W. H. Glanville, "The Creep or Flow of Concrete under Load," *Building Research Station Technical Paper No.* 12, H.M. Stationery Office, London.

¶ The simplified trapezoidal stress-strain diagram for concrete was first proposed by Dr. F. v. Emperger (Der Beiwert "*n*," *Beton und Eisen*, Vol. 35 (1936), p. 324-332). Its adaptation to ultimate strength design is due mainly to Prof. V. P. Jensen ("Ultimate Strength of Reinforced Concrete Beams as Related to the Plasticity of Concrete," *Illinois Univ. Eng. Exp. Sta. Bull.* No. 345, 1943). The method has been extended to combined bending and compression by the author ("Ultimate Strength Theory for Eccentrically Loaded Reinforced Concrete Columns," *Mag. of Conc. Res.*, Vol. 3 (1951), p. 19-22).

§ D. Ramsley and D. McHenry, "Stress-strain Curves for Concrete Strained beyond the Ultimate Load," *U.S. Bureau of Reclamation, Lab. Rep. No.* SP-12, Denver, 1947. H. J. Cowan, "Inelastic Deformation of Reinforced Concrete in relation to Ultimate Strength, *Engineering*, Vol. 174 (1952), p. 276-278.

‖ J. Tucker ("Effect of Length upon the Strength of Compression Test Specimens," *Proc. Am. Soc. Test. Mat.*, Vol. 46 (1946), p. 976-984) has proposed a statistical theory based on the conception that the strength of the concrete depends on the strength of its weakest part. The probability of the presence of a weak particle increases with the size of the specimen.

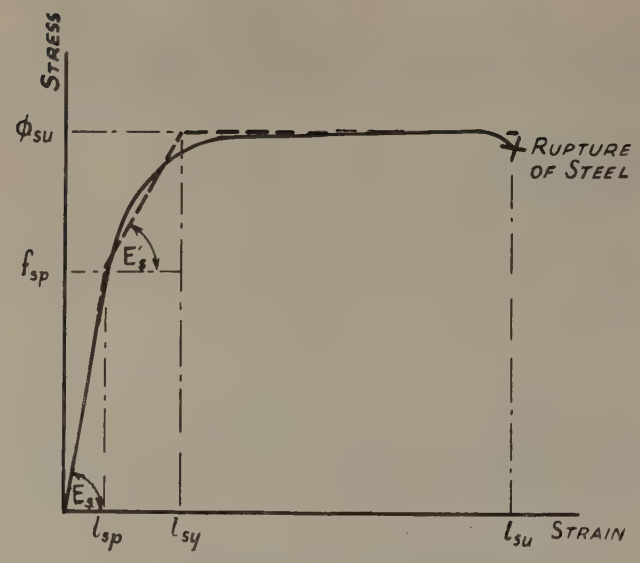


Fig. 2.—Actual and idealized stress-strain diagram for high tensile steel

varies from about 80 per cent. of the 6 in. cube crushing strength for low grade concrete to 95 per cent of the 6 inch cube crushing strength for concrete of very high strength. For concrete of the type normally used in prestressed beams

$$\varnothing_{cu} = 0.9\varnothing_{c6} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

may be considered a reasonable average value.

The ultimate compressive strain of concrete depends largely on the rate of loading. In addition low grade concretes are more compressible than those of high strength. For concrete of the type normally used in prestressed construction failing under the action of an excessive *live* load an ultimate compressive strain of 0.3 per cent. may be considered an average value.

The ratio of inelastic to total strain is termed the inelastic strain ratio

$$\beta = (e_{cu} - e_{cy}) / e_{cu} = (e_{cu} - \varnothing_{cu}/E_c) / e_{cu} \quad (2)$$

The value of β depends therefore on the ultimate compressive strain.

The high tensile steels used for prestressing concrete do not show the pronounced yield point familiar in mild steel. It is, however, possible to fix an equivalent yield strain, e_{sy}, above which the strains are mainly plastic (Fig. 2). For the prestressing steels commonly in use this equivalent yield point is of the order of 1 per cent.

Some plastic straining occurs below this limit, and the effective modulus of elasticity in the range of high stresses, E'_s, is therefore lower than the modulus of elasticity, E_s, below the limit of proportionality.

The ultimate steel strain, e_{su}, depends mainly on the diameter. For wires of the type used for post-tensioning it is of the order of 6 per cent. For the thinner wires normally used for pre-tensioning the ultimate strain is likely to be lower, and for high tensile bars it may be a little higher.

The stress-strain curve of prestressing steel may therefore be replaced by the approximate diagram indicated by the dotted line in Fig. 2. The steel is elastic, with a modulus E_s, for $o < e_s < e_{sp}$, where e_{sp} is the strain due to the prestressing force, making due allowance for loss of prestress. A lower effective modulus of elasticity E'_s is used for $e_{sp} < e_s < e_{sy}$. The steel is assumed to reach the ultimate tensile stress $\varnothing_{su}$, at a strain e_{sy}, and thereafter it becomes full plastic, until it fractures at a strain e_{su}. The stress of high tensile steel increases with increasing strain, even in the plastic range (Fig. 2) ; the stress in the wires or bars of a beam failing with a steel strain only a little greater than e_{sy} is therefore 10 to 20 per cent. lower than the ultimate stress. In practice it is convenient to use the ultimate tensile stress, $\varnothing_{su}$, obtained from the standard tensile test, in all computations of primary tension failures. This gives the true load factor for beams failing with steel rupture, but a nominal load factor which is somewhat lower than the actual load factor for beams failing with comparatively low plastic strains ; in view of the violent character of the former type of failure, this extra reserve in strength is evidently desirable.

For a beam in which yielding of the steel only just precedes crushing of the concrete, a nominal load factor of 2 would thus correspond to a true load factor of perhaps 1.85. The author considers this to be sufficient for prestressed concrete failing through yielding of the reinforcement, but without steel rupture. When a more accurate determination of the ultimate load is required, the actual stress diagram (shown in the full line in Fig. 2) must be used in place of the approximate diagram (shown dotted).*

Equivalent External Prestressing Force

A prestressed beam is subject to combined bending (due to the transverse loads) and compression (due to the prestressing force). As a result of loss due to shrinkage and creep (and pre-compression in pre-tensioned beams only) the initial prestressing force is reduced to P_f, the final prestressing force at the time when the beam is loaded to destruction. It is convenient to introduce an equivalent force P_e, secured against hypothetical external anchorages, which on release from these anchorages would produce the same stresses in the steel and the concrete as the force P_f. The concrete would thus be unstressed under the action of the force P_e. The steel stress due to P_e

$$f_{sp} = P_e/A_s \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

would be a little higher than the actual steel stress

$$f_{sf} = P_f/A_s \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Since the change in strain in the steel and in the surrounding concrete must be equal

$$(f_{sp} - f_{sf}) / E_s = K f_{cf} / E_c \quad . \quad . \quad . \quad (5)$$

where K is the strain factor (which is equal to unity when the steel is bonded to the concrete).

The stress in the concrete immediately surrounding the prestressing steel

$$f_{cf} = \frac{P_f}{A_c} + \frac{P_f \epsilon^2}{I_c} = \frac{P_f}{A_c}\left(1 + \frac{\epsilon^2}{k^2}\right)$$

where ϵ is the eccentricity of the prestressing force, I_c is the second moment of area of the concrete section, and k is its radius of gyration.

* E. Mörsch, "Die Ermittlung des Bruchmoments von Spannbetonträgern," *Beton und Stahlbetonbau*, Vol. 45 (1950), p. 149-157.

Substituting from equations (4) and (5)

$$\frac{f_{sp} - f_{st}}{K} \frac{E_c}{E_s} = f_{st} \frac{A_s}{A_c} \left(1 + \frac{\epsilon^2}{k^2}\right)$$

Putting $m = E_s/E_c$ and $r = A_s/A_c$, the equivalent external prestress

$$f_{sp} = f_{st} \left[1 + K m r (1 + \epsilon^2/k^2)\right] \quad . \quad (6)$$

This may be written as

$$f_{sp} = f_{st}/\theta' \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

where θ' is the equivalent transfer loss constant, allowing for loss due to pre-compression only. It is not generally necessary to determine the value of θ' in each instance; $\theta' = 0.95$ would be a conservative value for most beams.

The equivalent initial steel stress is approximately the same for both pre-tensioned and post-tensioned wires. Taking the initial steel stress for pre-tensioned wires as 160,000 lb. per sq. in., and allowing 15 per cent. for loss on transfer, and a further 10 per cent. for loss due to creep, $f_{st} = 122{,}500$ lb. per sq. in. Taking the initial steel stress for post-tensioned wires as 140,000 lb. per sq. in. and allowing 15 per cent. for loss due to creep, $f_{st} = 119{,}000$ lb. per sq. in. (for high tensile steel *bars* the final steel stress would be somewhat lower). Taking $\theta' = 0.95$, the equivalent initial steel stress is therefore of the order of 126,000 lb. per sq. in. The equivalent initial steel strain

$$e_{sp} = f_{sp}/E_s$$

taking the modulus of elasticity as 28×10^6 lb. per sq. in., is then approximately 0.45 per cent.

The Position of the Neutral Axis

Since the strains at failure are compressive at the top of the beam and tensile at the bottom, there is a neutral axis at which the strain changes from tension to compression. If it is assumed that plane sections remain plane after bending, the strain is proportional to the distance from the neutral axis, as shown in Fig. 3. The validity of this assumption has been demonstrated on numerous occasions by tests on normal reinforced concrete beams, and it is borne out by the available experimental data relating to prestressed beams.

If there is perfect bond, the change in the strain in the steel, e_s, and in the surrounding concrete is the same. If the steel is not bonded to the concrete, or if there is failure of bond, the strain in the surrounding concrete is e_s/K', where K' is the strain factor at the ultimate load. For pre-tensioned wires of small diameter K' may be taken equal to unity. For post-tensioned cables which are subsequently grouted K' is approximately 0.95. For grouted bars K' is likely to be between 0.85 and 0.90, depending on the size of the bar. Values of K' ranging from 0.1 to 0.4 have been observed for post-tensioned steel which is not bonded to the concrete.*

From Fig. 3 the relation between the concrete stress at the top face e_c, and the concrete strain at steel level

$$K' e_c/e_s = n/(d - n)$$

where n is the depth of the neutral axis, and d is the effective depth of the section measured to the centre of the steel.

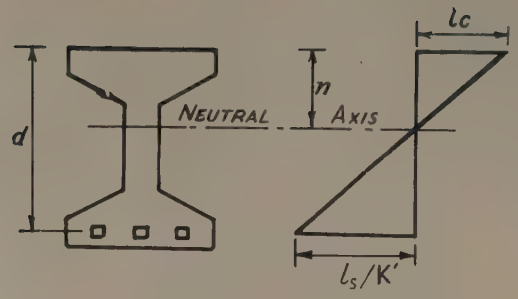


Fig. 3.—Position of neutral axis

The neutral axis depth ratio is therefore

$$n/d = K' e_c/(K' e_c + e_s) \quad . \quad . \quad . \quad . \quad (8)$$

The mode of failure is determined by the position of the neutral axis. If the steel reaches its ultimate strain, e_{su}, before the concrete at the top face reaches its ultimate strain, e_{cu}, failure occurs by rupture of the wires. (*Primary tension failure with steel rupture.*) At the upper limit of this mode of failure the steel and concrete reach their ultimate strains simultaneously, i.e. $e_c = e_{cu}$ and $e_s = e_{su} - e_{sp}$, where e_{sp} is the initial steel strain due to the equivalent external prestress.

The limiting neutral axis depth ratio for steel rupture is therefore

$$n_r/d = K' e_{cu}/(K' e_{cu} + e_{su} - e_{sp}) \quad . \quad . \quad (9)$$

Taking $e_{cu} = 5 \times 10^{-3}$†; $e_{su} = 60 \times 10^{-3}$; $e_{sp} = 4.5 \times 10^{-3}$; and $K' = 1$, for example, then $n_r = 0.084\,d$. If the strain factor, K', or the ultimate compression strain, e_{cu}, are lower, the limiting neutral axis depth is even less.

When the steel reaches its equivalent yield strain, e_{sy}, the steel stress reaches its maximum value (Fig. 2) and the resistance moment of the section can be increased only by lengthening the moment arm. The neutral axis therefore rises until the ultimate concrete strain is reached at the top face, when the beam fails through crushing of the concrete. Since the failure is initiated by excessive yielding of the steel, this is a *primary tension failure without steel rupture.* The upper limit of this mode of failure is reached when the steel reaches its yield strain and the concrete reaches its ultimate strain at the same load.

The limiting neutral axis depth ratio for primary tension failure is therefore

$$n_o/d = K' e_{cu}/(K' e_{cu} + e_{sy} - e_{sp}) \quad (10)$$

The limiting strains do not differ greatly for the steel and concrete commonly in use, but the strain factor may vary appreciably when the steel is not bonded to the concrete. The effect of this variation on the limiting neutral axis depth ratio for some typical limiting strains is shown in Table 1.

* A. L. L. Baker, "The Ultimate Strength in Bending of Prestressed Concrete Beams," *Reinforced Concrete Review*, Vol. 2 (July 1951), p. 343-365. D. W. Tollerfield, Thesis (*Unpublished*), *University of Sheffield*, 1953.

† This may be considered a comparatively high value for the ultimate concrete strain.

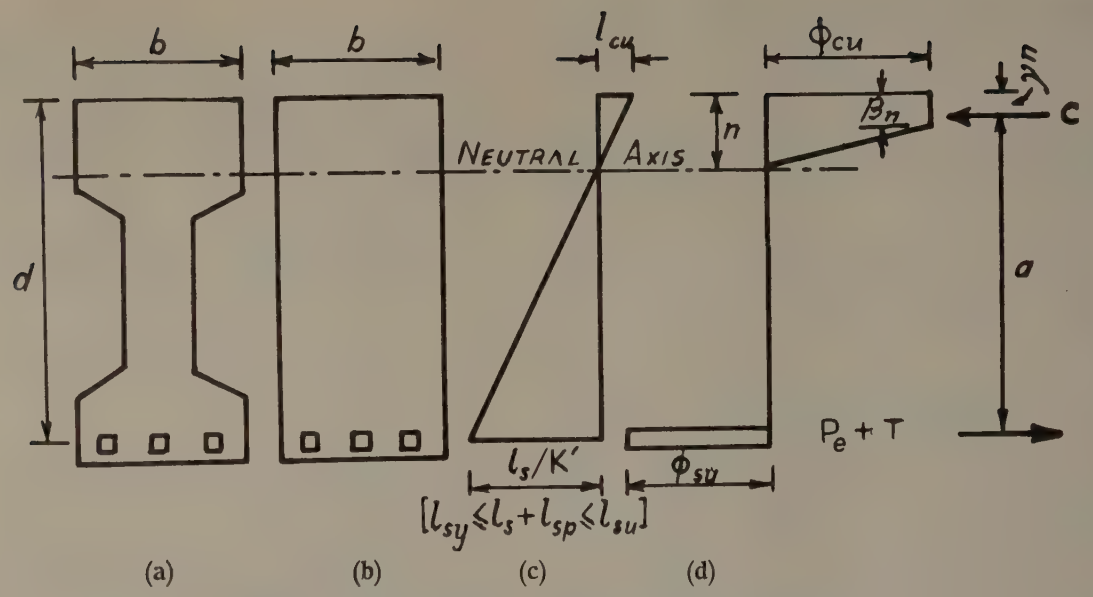


Fig. 4.—Primary tension failure without steel rupture

Table 1

Variation of limiting neutral axis depth, n_o, with strain factor, K'.

($e_{cu} = 3 \times 10^{-3}$* ; $e_{sy} = 10 \times 10^{-3}$; $e_{sp} = 4.5 \times 10^{-3}$)

K'	n_o/d
1.0	0.35
0.9	0.33
0.8	0.30
0.6	0.26
0.4	0.18
0.2	0.10
0.1	0.05

The slight reduction in the strain factor which may result from local bond failure does not therefore affect the value of n_o to any appreciable extent. A very considerable reduction in the limiting neutral axis depth ratio results, however, from the low strain factors which may occur in un-bonded post-tensioned beams. This in turn seriously reduces the ultimate strength of the more heavily reinforced unbonded beams.

When the concrete reaches its ultimate compressive strain before the steel reaches its equivalent yield strain, failure occurs through crushing of the concrete while the steel is still mainly elastic (*primary compression failure*). The total elongation of the steel is then comparatively small, and the ultimate deflection is reduced accordingly. There is therefore less warning of impending failure than in the case of a beam which shows appreciable yielding of the steel. The low deflection also reduces the capacity of the beam for absorbing energy, and consequently lowers its impact resistance.

Primary Tension Failure

If the steel strain at failure is greater than the equivalent yield strain, e_{sy}, the steel stress at failure (using the approximate diagram shown dotted in Fig. 2) is equal to the ultimate tensile strength, $\varnothing_{su}$. Failure occurs gradually with appreciable ultimate deflection. The great majority of practical prestressed concrete bean would, in fact, show this type of failure if tested destruction.

Fig. 4 shows the strain and stress distribution fi failure without steel rupture. The steel reaches ultimate tensile strength, $\varnothing_{su}$, under the combin action of the equivalent external prestressing force, P and of the additional tension due to the ultimate ben ing moment, T.

$$P_e + T = \varnothing_{su} A_s \quad . \quad . \quad . \quad . \quad . \quad (1$$

where A_s is the cross-sectional area of the steel.

The concrete reaches its ultimate compressive stra at the top face, and the resultant compressive force therefore represented by the area of the trapezium Fig. 4 (d).

$$C = \varnothing_{cu} b(\beta n + \tfrac{1}{2}(n - \beta n)) = \tfrac{1}{2}\varnothing_{cu} b\, n\, (1+\beta) \quad (1$$

where $\varnothing_{cu}$ is the ultimate compressive strength the concrete ; β is its inelastic strain ratio (equal $(e_{cu} - e_{cy})/e_{cu}$) ; n is the depth of the neutral axi and b is the width of the rectangular section, or t width of the top flange of a box, T, or I section whe the neutral axis is inside the flange.

For horizontal equilibrium

$$C = P_e + T \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1.$$

and therefore

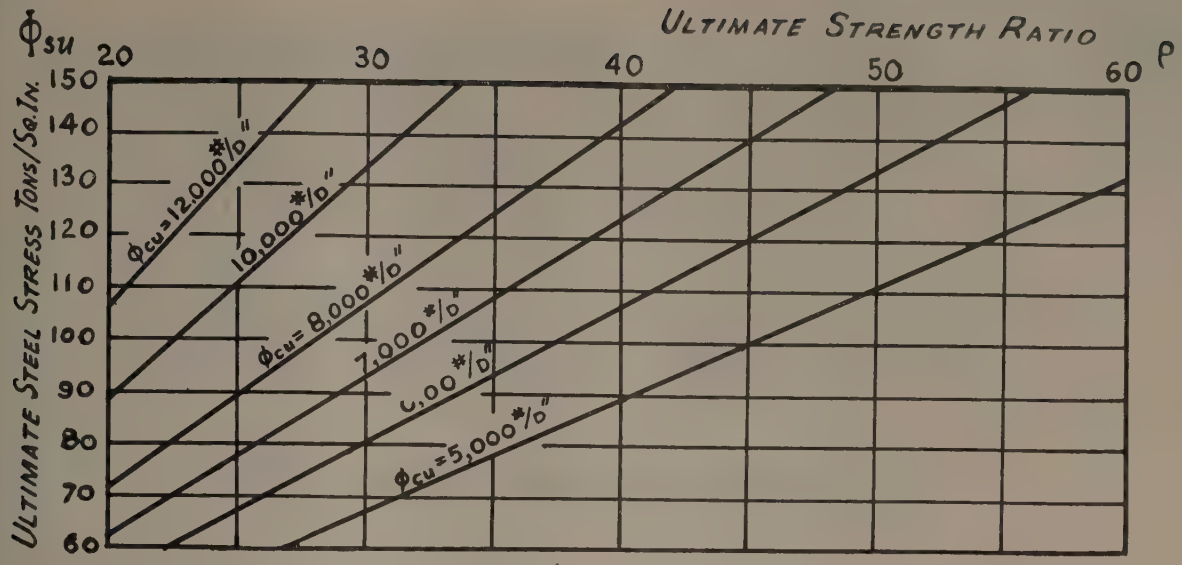
$$\tfrac{1}{2}\varnothing_{cu}\, b\, n\, (1 + \beta) = \varnothing_{su} A_s$$

Substituting $\rho = \varnothing_{su}/\varnothing_{cu}$ (ultimate strength rati and $r' = A_s/bd$ (steel ratio)† the neutral axis dept ratio

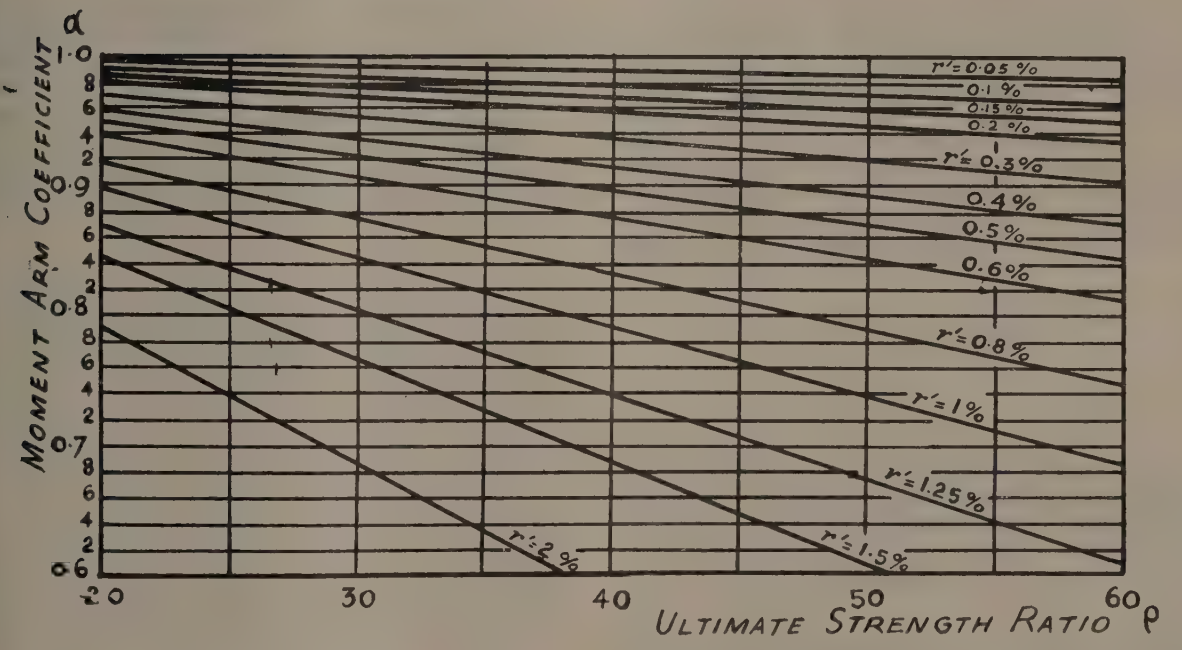
$$n/d = 2\rho r'/(1 + \beta) \quad . \quad . \quad . \quad . \quad . \quad (1.$$

* This may be considered a comparatively low value for t ultimate concrete strain.

† The steel ratio r' generally has a much lower numerical val than the steel ratio $r = A_s/A_c$ used for design at working load



(*a*) **Ultimate strength ratio** (ρ)



(*b*) **Moment arm coefficient** (α) **for primary tension failure (for inelastic strain ratio** $\beta = 0.50$)

Fig. 5.

For primary tension failure n must be smaller than n_o, the limiting neutral axis depth for primary compression failure.

If the neutral axis depth is less than n_r, the limiting depth for steel rupture, the beam fails through fracture of the wires before the concrete crushes. This happens in lightly reinforced beams, particularly when the steel is bonded and the concrete has a high ultimate strain. From equation (14) the minimum steel ratio required to prevent rupture of the wires

$$r'_r = \tfrac{1}{2}(1 + \beta)\, n_r/d\rho$$

The beam will show primary tension failure if the neutral axis depth is less than n_o, the limiting depth for primary compression failure. The highest steel ratio for primary tension failure is therefore

$$r'_o = \tfrac{1}{2}(1 + \beta)\, n_o/d\rho$$

Taking, for example, $n_o/d = 0.35$ (corresponding to $K' = 1$ in Table 1), $\beta = 0.5$ and $\rho = 35$, then $r'_o = 0.0075$ ($\frac{3}{4}$ per cent.). If the steel is not bonded, primary compression failure may occur with very low steel ratios, due to the reduction in the strain factor. Taking $K' = 0.1$, for example, n_o/d is reduced to 0.05, and r'_o is then 0.0011 ($\frac{1}{10}$ per cent.).

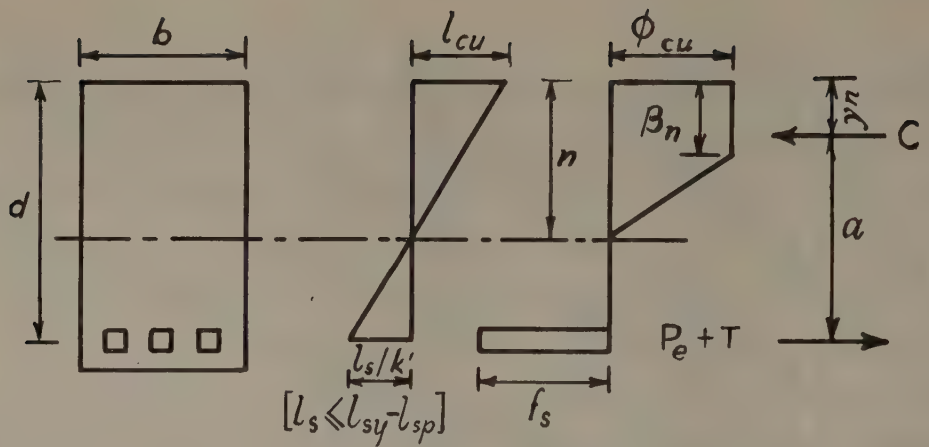


Fig. 6.—Primary compression failure of rectangular section

The length of the moment arm coefficient is obtained by taking moments about the compression face

$$C\,(d-a) = \varnothing_{cu}\, b\, \beta n \;.\; \tfrac{1}{2}\beta n + \tfrac{1}{2}\varnothing_{cu}\, b\,(1-\beta)n \;.\; (\beta n + \tfrac{1}{3}(1-\beta)n)$$

Substituting for C from equation (12)

$$\tfrac{1}{2}\varnothing_{cu}\, bn\,(1+\beta)\;.\;(d-a) = \tfrac{1}{6}\varnothing_{cu}\, b\, n^2\,(1+\beta+\beta^2)$$

which gives the moment arm

$$a = d - \tfrac{1}{3}\,n\,(1+\beta+\beta^2)/(1+\beta) = d-\gamma n \quad . \qquad (15)$$

where $\gamma = (1+\beta+\beta^2)/3\,(1+\beta)$.

The moment arm coefficient

$$\alpha = a/d = 1-\gamma n/d \quad . \quad . \quad . \quad . \quad . \quad (16)$$

If the beam fails by steel rupture, the concrete does not reach its ultimate compressive strain, and the compressive stress block in Fig. 4(d) is therefore approximately triangular in shape. The length of the moment arm coefficient is increased accordingly. The increase is, however, negligible due to the very low value of the neutral axis depth.

The ultimate strength ratio, ρ, may be obtained from the upper part of Fig. 5. The variation of the moment arm coefficient with the steel ratio r' is shown in the lower part of the diagram for an inelastic strain ratio of 0.50. In I, T, and box sections the steel ratio r' is usually low, and the moment arm coefficient is mostly greater than 0.90.

Taking moments about the line of action of the compressive force C, the ultimate resistance moment of the section

$$M_u = (P_e+T)\,a = \varnothing_{su}\, A_s\, a = \varnothing_{su}\, A_s\, \alpha\, d \qquad (17)$$

where a is the length of the moment arm, and α the moment arm coefficient.

The load factor against failure is defined by

$$L_u = M_u/(M_D+M_L) \quad . \quad . \quad . \quad . \quad (18)$$

where M_D and M_L are the bending moments due to the dead weight of the beam and due to the superimposed loads respectively.

Primary Compression Failure of Rectangular Sections

If the steel strain at failure is less than the equivalent yield strain, e_{sy}, the concrete crushes while the steel is still mainly elastic, and the ultimate deflection is th comparatively small.

The steel stress under the combined action of t equivalent external prestressing force, P_e, and t additional tension due to the ultimate bending momer T, is then less than the ultimate stress.

$$(P_e+T)/A_s = f_{sp}+e_s\, E'_s \quad . \quad . \quad . \quad . \quad (1$$

where f_{sp} is the steel stress due to the prestressing fore e_s is the steel strain due to the bending moment, a E'_s is the effective modulus of elasticity of the steel high loads (Fig. 2).

If plane sections remain plane (Fig. 6)

$$e_s/K' = e_{cu}\,(d-n)/n \quad . \quad . \quad . \quad . \quad . \quad (2$$

where e_{cu} is the concrete strain at the top of the bea (which is equal to the ultimate strain when the concre crushes), and K' is the strain factor.

Using the simplified stress-strain diagram for co crete shown dotted in Fig. 1

$$e_{cy} = (1-\beta)\, e_{cu} = \varnothing_{cu}/E_c$$

where $\varnothing_{cu}$ is the ultimate concrete strength, and β the inelastic strain ratio. Substituting this expressi in equation (20)

$$e_s = \frac{K'\varnothing_{cu}}{E_c\,(1-\beta)}\;\frac{d-n}{n}$$

Putting $m' = E'_s/E_c\;(1-\beta)$ (equivalent modul ratio at high loads), and substituting into equation (1

$$P_e+T = f_{sp}\, A_s + m'K'\varnothing_{cu}\, A_s\,(d-n)/n \quad (2$$

The resultant compressive force (computed pr viously)

$$C = \tfrac{1}{2}\varnothing_{cu}\, b\, n\,(1+\beta) \quad . \quad . \quad . \quad . \quad . \quad (1$$

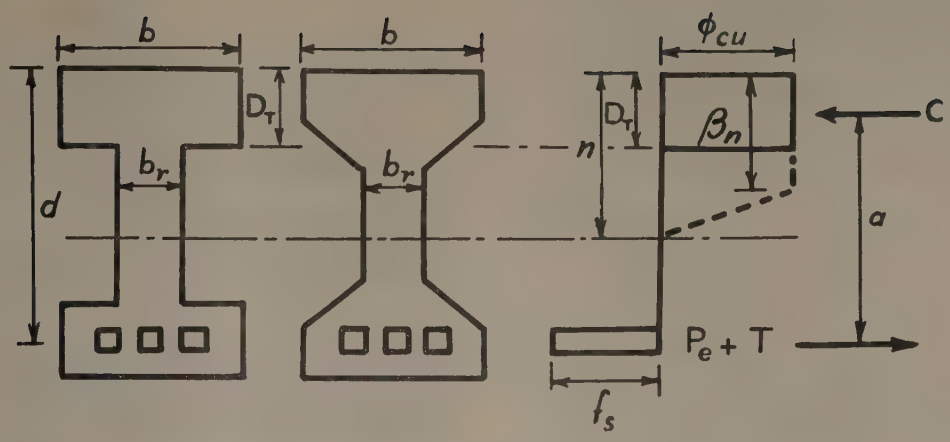


Fig. 7.—Primary compression failure of flanged section

For horizontal equilibrium

$$C = P_e + T \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

Substituting from equations (13) and (21)

$$\tfrac{1}{2} \varnothing_{cu}\, b\, n\, (1+\beta) + m' K' \varnothing_{cu}\, A_s - f_{sp}\, A_s = m' K' \varnothing_{cu}\, A_s \frac{d}{n}$$

Dividing by $\frac{1}{2} \varnothing_{cu}\, b\, d^2\, (1+\beta)/n$, and substituting $r' = A_s/b\, d$ a quadratic equation for the neutral axis depth ratio is obtained.

$$(n/d)^2 + (2r'/(1+\beta)) \cdot (m'K' - f_{sp}/\varnothing_{cu})\, n/d = 2\, m' K' r'/(1+\beta) \quad . \quad . \quad . \quad . \quad . \quad . \quad (22)^*$$

The depth of the neutral axis evidently depends on the magnitude of the prestress and on the strain factor. A high prestress increases the neutral axis depth. This in turn increases the area of the compression zone, and consequently raises the ultimate resistance moment. A low strain factor decreases the neutral axis depth, and this reduces the ultimate resistance moment.

Taking moments about the centre of the prestressing steel, the ultimate resistance moment

$$M_u = C\, a$$

Substituting from equations (12) and (15)

$$M_u = \tfrac{1}{2} \varnothing_{cu}\, b\, n\, (1+\beta) \cdot (d - \gamma n) \quad . \quad (23)$$

where $\gamma = (1+\beta+\beta^2)/3\,(1+\beta)$

Primary Compression Failure of Flanged Sections

If compressive stresses in the concrete rib are neglected, as is customary in normal reinforced concrete T-beams, the resultant compressive force (Fig. 7)

$$C = \varnothing_{cu}\, b\, D_T \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (24)$$

provided

$$n \geq D_T/\beta \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (25)$$

where D_T is thickness of the top flange. The depth of the neutral axis is determined by ratio of the strains in the steel and the concrete at the top face.

When the steel is not bonded to the concrete, the strain factor is generally low, and the neutral axis is likely to lie inside the top flange. The flanged section is then equivalent to a rectangular section, and may be designed by the equations derived in the previous section.

When the steel is bonded to the concrete, the neutral axis depth (from Table 1) is approximately 0.35 d. Taking the inelastic strain ratio $\beta = 0.5$, equation (25) is satisfied if $D_T/d < 0.18$, which is true for most flanged sections.

The compressive stress bock is therefore rectangular in shape, and the moment arm is

$$a = d - \tfrac{1}{2} D_T \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (26)$$

The ultimate resistance moment of the section (neglecting compression in the rib), taking moments about the centre of the steel, is therefore

$$M_u = C\, a = \varnothing_{cu}\, b\, D_T\, (d - \tfrac{1}{2} D_T) \quad . \quad . \quad (27)$$

If the section has a comparatively thick rib and the neutral axis is considerably below the bottom of the flange, a more accurate estimate of the ultimate resistance moment must be made by including the compressive stresses in the concrete rib.

The resultant compressive force is then (Fig. 7)

$$C = \varnothing_{cu}\, (\tfrac{1}{2}\, b_r\, n\, (1+\beta) + (b - b_r)\, D_T) \quad . \quad (28)$$

where b_r is the thickness of the rib.

The total force in the steel (computed previously)

$$P_e + T = f_{sp}\, A_s + m' K' \varnothing_{cu}\, A_s\, (d - n)/n \quad (21)$$

For horizontal equilibrium

$$C = P_e + T \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

* This equation is similar in construction to the well-known equation for determining the position of the neutral axis in normal reinforced concrete beams $(n/d)^2 + 2\, m\, r'\, n/d = 2\, m\, r'$

Substituting from equations (21) and (28)

$$\tfrac{1}{2}\varnothing_{cu}\, b_r\, n\,(1+\beta) + m'K'\varnothing_{cu}\, A_s - f_{sp}\, A_s + \varnothing_{cu}\, b\, D_T - \varnothing_{cu}\, b_r\, D_T = m'K'\varnothing_{cu}\, A_s\, n/d$$

Dividing by $\tfrac{1}{2}\varnothing_{cu}\, b_r\, d^2\ (1+\beta)/n$, and substituting $r' = A_s/b\,d$, the quadratic equation for the neutral axis depth ratio is then

$$(n/d)^2 + \left[2/(1+\beta)\right] \cdot \left[b/b_r\,(m'K'r' - r' f_{sp}/\varnothing_{cu} + D_T/d) - D_T/d\right] n/d = 2\,m'K'r'\, b/b_r\,(1+\beta) \quad . \quad . \quad . \quad (29)$$

provided $n \geq D_T/\beta$

Taking moments about the centre of the steel separately for each component part of equation (28), the ultimate resistance moment of the section

$$M_u = C\,a = \varnothing_{cu} \left[\tfrac{1}{2}\, b_r\, n\,(1+\beta)\,.\,(d - \gamma n) + (b - b_r)\, D_T\,(d - \tfrac{1}{2}D_T)\right] \quad (30)$$

where n is the neutral axis depth obtained from equation (29), and

$$\gamma = (1+\beta+\beta^2)/3(1+\beta).$$

If the section is of complex shape, and the portion above the neutral axis cannot be approximated with sufficient accuracy to a rectangular flange and a rectangular rib, the ultimate resistance moment can be obtained by a graphical method devised by Prof. Mörsch.* The solution is obtained by a series of successive approximations, and is therefore rather laborious.

Effect on the Ultimate Strength of Prestressing Steel near the Compression Face

Primary Tension Failure

The prestressing steel is sometimes placed in two layers, one near each face of the beam. The upper layer of prestressing steel produces compressive stresses near the top face of the section which combine with the compressive stresses due to the bending moment to lower the ultimate strength of the beam. This effect is comparatively slight in beams showing primary tension failure,† but may become appreciable in beams with sufficient steel near the bottom face to cause primary compression failure.

Fig. 8 shows the strain and stress distribution at failure. The steel in the bottom layer reaches the ultimate tensile strength, $\varnothing_{su}$, under the combined action of the equivalent external lower prestressing force, P_{eB}, and the additional tensile force due to the bending moment, T.

$$P_{eB} + T = \varnothing_{su}\, A_{sB} \quad . \quad . \quad . \quad . \quad . \quad (31)$$

where A_{sB} is the area of the bottom steel.

The stress in the top layer of steel due to the equivalent external upper prestressing force, P_{eT}, is

$$f_{sp} = P_{eT}/A_{sT} \quad . \quad . \quad . \quad . \quad . \quad . \quad (32)$$

where A_{sT} is the area of the top steel. Due to its proximity to the neutral axis, this stress remains virtually unchanged when the load is applied.

The resultant compressive force is therefore entirely carried by the concrete above the neutral axis, and is represented by the area of the trapezium in Fig. 8(c).

$$C = \tfrac{1}{2}\varnothing_{cu}\, b\, n\,(1+\beta) \quad . \quad . \quad . \quad . \quad (12)$$

* E. Mörsch, "Die Ermittlung des Bruchmoments von Spannbetonträgern," *Die Bautechnik*, Vol. 26 (1949), p. 98-99.

† If the neutral axis is *above* the centre of the top steel, the latter acts as additional tension reinforcement, and the ultimate strength is *increased* accordingly.

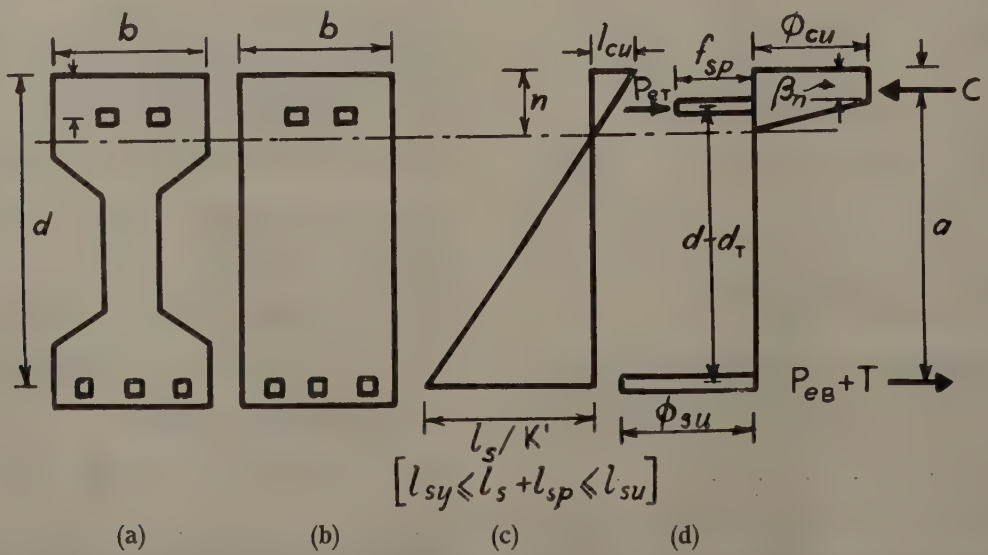


Fig. 8.—Primary tension failure without steel rupture of sections with double reinforcement

For horizontal equilibrium

$$C = P_{eT} + T + P_{eB} \quad . \quad . \quad . \quad . \quad (33)$$

Substituting from equations (12), (31) and (32)

$$\tfrac{1}{2}\varnothing_{cu}\, b\, n\, (1+\beta) = \varnothing_{su}\, A_{sB} + f_{sp}\, A_{sT}$$

Dividing by $\tfrac{1}{2}\varnothing_{cu}\, b\, d\, (1+\beta)$, and substituting $r'_T = A_{sT}/bd$ (top steel ratio) ; $r'_B = A_{sB}/bd$ (bottom steel ratio) ; $\rho = \varnothing_{su}/\varnothing_{cu}$ (ultimate strength ratio) and $\rho_s = \varnothing_{su}/f_{sp}$; the neutral axis depth ratio

$$n/d = 2\rho\, (r'_B + r'_T/\rho_s)/(1+\beta) \quad . \quad . \quad (34)$$

The ratio ρ_s depends essentially on the maximum permissible steel stress.*

Taking moments about the line of action of the force C, the ultimate resistance moment

$$\begin{aligned} M_u &= (P_{eB}+T)\ .\ (d-\gamma n) + P_{eT}\,(d_T-\gamma n) \\ &= \varnothing_{su}\,(A_{sB}\,a + A_{sT}\,(d_T-\gamma n)/\rho_s) \quad . \quad (35) \end{aligned}$$

where a is the length of the moment arm corresponding to an equivalent steel ratio $r' = r'_B + \tfrac{1}{2}r'_T$, which can be obtained from Fig. 6 ; d_T is the distance of the top steel from the compression face ; and $\gamma = (1+\beta+\beta^2)/3(1+\beta)$.

The distance $d_T - \gamma n$ may be either positive or negative ; it is generally very small due to the proximity of the neutral axis to the top steel. The second term can therefore be neglected in many practical problems, and

$$M_u = \varnothing_{su}\, A_{sB}\, a = \varnothing_{su}\, A_{sB}\, \alpha\, d \quad . \quad . \quad (36)$$

gives a good approximation which is generally on the safe side. Equation (36) is identical with equation (17) for primary tension failure of beams with bottom steel only; the length of the moment arm is, however, reduced by the presence of the top steel.

Primary Compression Failure of Rectangular Sections

The reduction in ultimate strength due to the top steel is more pronounced in sections with sufficient bottom steel to cause primary compression failure.

The bottom steel stress under the combined action of the equivalent external prestressing force, P_{eB}, and the additional tensile force due to the ultimate bending moment, T, is then less than the ultimate stress.

$$(P_{eB}+T)/A_{sB} = f_{sp} + e_{sB}\, E'_s < \varnothing_{su} \quad . \quad (37)$$

where f_{sp} is the steel stress due to the prestressing force ; e_{sB} is the steel strain due to the bending moment ; and E'_s is the effective modulus of elasticity of the steel at high loads (Fig. 2).

The application of the superimposed load reduces the stress in the top steel.

$$(P_{eT}-C_s)/A_{sT} = f_{sp} - e_{sT}\, E'_s \quad . \quad . \quad (38)$$

where C_s is the reduction in the force in the top steel due to the bending moment.

Since it is assumed that plane sections remain plane

$$e_{cu}/n = e_{sT}/K'_T\,(n-d_T) = e_{sB}/K'_B\,(d-n) \quad (39)$$

where K'_T and K'_B are the strain factors for the top and bottom steel respectively (defined as the ratio of the change of steel strain to the strain in the surrounding concrete). These are equal to unity when there is perfect bond between the two materials.

Using the simplified stress-strain diagram for concrete shown dotted in Fig. 1,

$$e_{cy} = (1-\beta)\, e_{cu} = \varnothing_{cu}/E_c$$

Substituting this expression into equation (39)

$$e_{sB} = \frac{K'_B \varnothing_{cu}}{E_c\,(1-\beta)}\ \frac{d-n}{n} \quad . \quad . \quad . \quad (40)$$

and

$$e_{sT} = \frac{K'_T \varnothing_{cu}}{E_c\,(1-\beta)}\ \frac{n-d_T}{n} \quad . \quad . \quad . \quad (41)$$

Putting $m' = E'_s/E_c\ (1-\beta)$ (equivalent modular ratio at high loads), and substituting into equations (37) and (38)

$$P_{eB} + T = f_{sp}\, A_{sB} + m'K'_B \varnothing_{cu}\, A_{sB}\,(d-n)/n \quad (42)$$

and

$$P_{eT} - C_s = f_{sp}\, A_{sT} - m'K'_T \varnothing_{cu}\, A_{sT}\,(n-d_T)/n \quad (43)$$

The compressive force taken by the concrete

$$C_c = \tfrac{1}{2}\varnothing_{cu}\, b\, n\, (1+\beta) \quad . \quad . \quad . \quad . \quad (44)$$

For horizontal equilibrium

$$C_c + C_s = P_{eT} + P_{eB} + T \quad . \quad . \quad . \quad (45)$$

Substituting from equations (42) to (44)

$$\begin{aligned} &\tfrac{1}{2}\varnothing_{cu}\, bn\,(1+\beta) + m'K'_B\varnothing_{cu}\, A_{sB} \\ &\quad + m'K'_T\varnothing_{cu}\, A_{sT} - f_{sp}\, A_{sB} - f_{sp}\, A_{sT} \\ &\quad = m'K'_B\varnothing_{cu}\, A_{sB}\, d/n + m'K'_T\varnothing_{cu}\, A_{sT}\, d_1/n \end{aligned}$$

Dividing by $\tfrac{1}{2}\varnothing_{cu}\, b\, d^2\ (1+\beta)/n$, and substituting $r'_B = A_{sB}/bd$ and $r'_T = A_{sT}/bd$, a quadratic equation for the neutral axis depth ratio is obtained.

* In pre-tensioned beams the maximum permissible initial prestress is approximately 65 per cent. of the ultimate tensile strength. Allowing 10 per cent. for loss due to shrinkage, and a further 10 per cent. for loss due to creep

$$f_{sp} = 0.65 \times 0.9 \times 0.9 \times \varnothing_{su}.$$

(The loss due to pre-compression is recovered, since f_{sp} is the hypothetical final prestress due to an equivalent *external* force P_e).

The ratio

$$\rho_s = \varnothing_{su}/f_{sp} = 1.90$$

For post-tensioned beams the maximum permissible initial prestress is approximately 60 per cent. of the ultimate tensile strength. Allowing 15 per cent. for loss due to creep and shrinkage, and 5 per cent. for the increase in the steel stress which would be required to remove the compressive stresses in the concrete (recovery of precompression)

$$f_{sp} = 0.60 \times 0.85 \times 1.05 \times \varnothing_{su}$$

which gives

$$\rho_s = \varnothing_{su}/f_{sp} = 1.87.$$

If the initial steel stresses are a little lower, the value of ρ_s is increased accordingly. In determining the ultimate resistance moment of doubly prestressed beams for primary tension failure, the magnitude of the ratio ρ_s is required only for determining the length of the moment arm, and a round value $\rho_s = 2$ should be sufficiently accurate for most purposes.

The depth of the neutral axis of a section with a top steel ratio r'_T and a bottom steel ratio r'_B is therefore virtually the same as that of a section with a bottom steel ratio $r'_B + \tfrac{1}{2}r'_T$. The length of the moment arm is reduced accordingly.

$$(n/d)^2 + \left[2/(1+\beta)\right] \cdot \left[(K'_B r'_B + K'_T r'_T)\, m' - (r'_B + r'_T) f_{sp}/\varnothing_{cu}\right] n/d$$
$$= 2\, m'(K'_B r'_B + K'_T r'_T d_T/d)/(1+\beta) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

(This reduces to equation (22) when $r'_T = 0$.)

Taking moments about the centre of the bottom steel, the ultimate resistance moment

$$M_u = C_c\, a - (P_{eT} - C_s) \cdot (d - d_T)$$
$$= \tfrac{1}{2}\varnothing_{cu}\, b\, n\, (1+\beta) \cdot (d - \gamma n) - \left[f_{sp} - m' K'_T \varnothing_{cu}(n - d_T)/n\right] A_{sT}(d - d_T) \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Although the depth of the compression zone (equal to the neutral axis depth, n) is increased by the top st this is offset by the negative term introduced into the ultimate resistance moment. The change in the top st stress is generally small by comparison with the initial prestress, and it can be neglected as a first approximat (which is always on the safe side).

$$M_u = \tfrac{1}{2}\varnothing_{cu}\, b\, n\, (1+\beta) \cdot (d - \gamma n) - f_{sp}\, A_{sT}\, (d - d_T) \cdot \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

Primary Compression Failure of Flanged Sections

If compressive stresses in the rib are ignored, the compressive force taken by the concrete (Figs. 7 and

$$C_c = \varnothing_{cu}\, b\, D_T \quad . \quad (4)$$

provided $n \leq D_T/\beta$

For horizontal equilibrium

$$C_c + C_s = P_{eT} + P_{eB} + T \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Substituting from equations (42), (43) and (49)

$$\varnothing_{cu}\, b D_T + m' K'_B \varnothing_{cu}\, A_{sB} + m' K'_T \varnothing_{cu}\, A_{sT} - f_{sp}\, A_{sB} - f_{sp}\, A_{sT}$$
$$= m' K'_B \varnothing_{cu}\, A_{sB}\, d/n + m' K'_T \varnothing_{cu}\, A_{sT}\, d_T/n$$

Dividing by $\varnothing_{cu}\, b\, d^2/n$, and substituting $r'_B = A_{sB}/bd$ and $r'_T = A_{sT}/bd$, a linear equation for the neutral ax depth ratio is obtained.

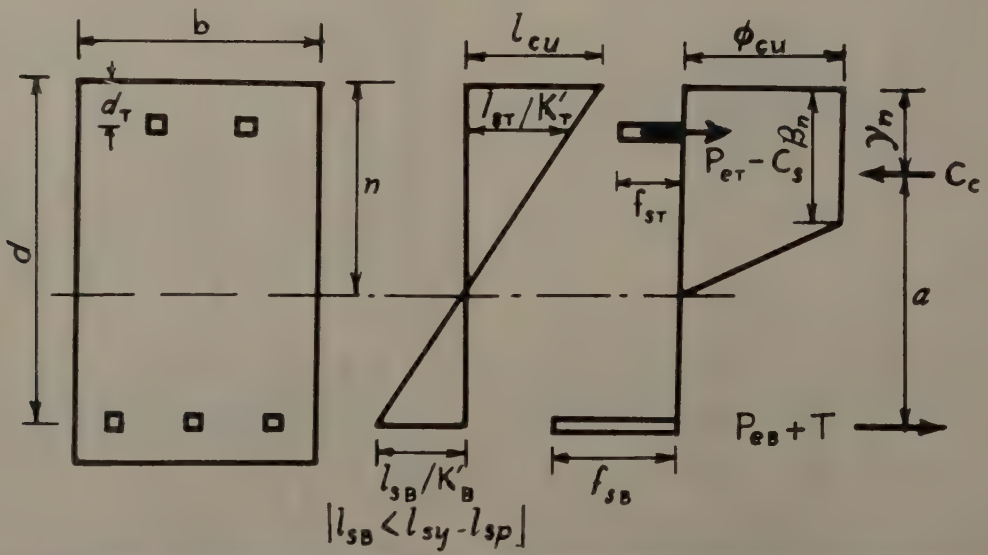


Fig. 9.—Primary compression failure of rectangular section with double reinforcement

$$\left[D_T/d + (K'_B r'_B + K'_T r'_T)m' - (r'_B + r_T')f_{sp}/\varnothing_{cu}\right] n/d = m'(K'_B r'_B + K'_T r'_T d_T/d) \quad . \quad . \quad . \quad . \quad (50)$$

Taking moments about the centre of the bottom steel, the ultimate resistance moment

$$M_u = \varnothing_{cu}\, b\, D_T\,(d - \tfrac{1}{2}D_T) - \left[f_{sp} - m'K'_T\varnothing_{cu}(n - d_T)/n\right] A_{sT}(d - d_T) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (51)$$

Since the depth of the compression zone is equal to the thickness of the flange when the compression in the web is ignored, the reduction in the ultimate resistance moment due to the addition of the top steel (represented by the second term in equation (51)) may be quite considerable. The change in the top steel stress is generally small in comparison with the initial steel stress, and it may be neglected as a first approximation (which is always on the safe side).

$$M_u = \varnothing_{cu}\, b\, D_T\,(d - \tfrac{1}{2}D_T) - f_{sp}\, A_{sT}\,(d - d_T). \quad (52)$$

Application of the Ultimate Strength Theory to Practical Design

Example 1. Calculate the ultimate resistance moment of a rectangular concrete section, 12 in. wide by 18 in. deep, prestressed by two $\frac{3}{4}$ in. diameter bars, placed at a distance of 2 in. from the tension face, (*a*) if the ducts are grouted after the completion of the tensioning process, and (*b*) if the bars are not bonded to the concrete.

It is assumed that the concrete has an ultimate compressive strength $\varnothing_{cu} = 6{,}000$ lb. per sq. in. (corresponding approximately to a cube crushing strength of 3,700 lb. per sq. in.) ; a modulus of elasticity $E_c = 4 \times 10^6$ lb. per sq. in. ; and an ultimate compressive strain $e_{cu} = 3 \times 10^{-3}$. From equation (2) the inelastic strain ratio is then

$$\beta = (3 \times 10^{-3} - 6{,}000/4{,}000{,}000)/3 \times 10^{-3} = 0.50$$

and the constant

$$\gamma = (1 + 0.50 + 0.25)/3(1 + 0.50) = 0.39$$

It is assumed that the steel is initially stressed to 100,000 lb. per sq. in. Allowing 15 per cent. for loss due to creep etc. and taking the equivalent transfer loss constant $\theta' = 0.95$, the equivalent external prestress

$$f_{sp} = 0.85 \times 100{,}000/0.95 = 89{,}400 \text{ lb. per sq. in.}$$

Taking the modulus of elasticity as 28×10^6 lb. per sq. in., the corresponding strain $e_{sp} = 3.2 \times 10^{-3}$. It is further assumed that the equivalent yield strain $e_{sy} = 9 \times 10^{-3}$, and the ultimate tensile strength $\varnothing_{su} = 70$ tons per sq. in. = 157,000 lb. per sq. in.

The effective modulus of elasticity of the steel

$$E'_s = (157{,}000 - 89{,}400)/(9 - 3.2) \times 10^{-3} = 11{,}600{,}000 \text{ lb. per sq. in.}$$

and the effective modular ratio

$$m' = 11.6 / 4 \times 0.50 = 5.8$$

The ultimate strength ratio

$$\rho = 157{,}000 / 6{,}000 = 26.2.$$

It is assumed that the strain factor is $K' = 0.90$ when the bars are grouted, and 0.25 when the bars are not grouted.

The dimensions of the section are

$b = 12$ in. ; $d = 18 - 2 = 16$ in. ;
$A_s = 0.883$ sq. in. ; and $r' = 0.883/12 \times 16 = 0.00460$.

(*a*) *Ducts grouted* ($K' = 0.90$)

The greatest neutral axis depth ratio for primary tension failure (from equation (10))

$$n_o/d = 0.90 \times 3 \times 10^{-3}/ (0.90 \times 3 + 9 - 3.2) \times 10^{-3} = 0.318.$$

From equation (14) the neutral axis depth ratio for the section

$$n/d = 2 \times 26.2 \times 0.00460/1.50 = 0.161.$$

This is less than n_o/d, and failure is therefore initiated by yielding of the steel. From equation (15) the moment arm coefficient

$$\alpha = 1 - 0.39 \times 0.161 = 0.937.$$

The ultimate resistance moment (from equation (17))

$$M_u = 157{,}000 \times 0.883 \times 0.937 \times 16 = 2{,}080{,}000 \text{ lb. in.}$$

(*b*) *Bars not bonded to concrete* ($K' = 0.25$).

The greatest neutral axis depth ratio for primary tension failure (from equation (10))

$$n_o/d = 0.25 \times 3 \times 10^{-3}/ (0.25 \times 3 + 9 - 3.2) \times 10^{-3} = 0.114$$

This is much lower than the value obtained in the previous paragraph, and failure is thus likely to be caused by crushing of the concrete. The neutral axis depth is therefore obtained from equation (22).

$$(n/d)^2 + (2 \times 0.00460/1.50) \times (5.8 \times 0.25 - 89{,}000/6{,}000)\, n/d = 2 \times 5.8 \times 0.25 \times 0.00460/1.50$$

This gives the neutral axis depth ratio

$$n/d = 0.0411 + \sqrt{(0.00170 + 0.00888)} = 0.144$$

which is, in fact, greater than n_o/d.

The neutral axis depth, $n = 2.30$ in., and the moment arm $a = 16 - 0.39 \times 2.30 = 15.1$ in.

From equation (23) the ultimate resistance moment for primary compression failure

$$M_u = \tfrac{1}{2} \times 6{,}000 \times 12 \times 2.3 \times 1.5 \times 15.1 = 1{,}880{,}000 \text{ lb. in.}$$

Example 2. Calculate the ultimate resistance moment of a rectangular concrete section, 12 in. wide by 18 in. deep, uniformly prestressed by four $\frac{3}{4}$ in. diameter bars, (*a*) if the bars are grouted after completion of the tensioning process, and (*b*) if the bars are not bonded to the concrete. The bars are placed in pairs at a distance of 2 in. from the top and bottom faces.

The properties of the materials are the same as in Example 1. The strain factor for the top steel is assumed as equal to 0.40 when the ducts are not grouted.

The dimensions of the section are

$$b = 12\text{ in.};\ d = 18 - 2 = 16\text{ in.};\ d_T = 2\text{ in.};$$
$$d_T/d = 0.125;\ A_{sB} = A_{sT} = 0.883\text{ sq. in.};$$
$$r'_B = r'_T = 0.00460.$$

(*a*) *Ducts grouted.* ($K'_B = K'_T = 0.90$.)

The ratio $\rho_s = 157{,}000/89{,}000 = 1.76$ and the equivalent steel ratio for the section

$$r'_B + r'_T/\rho_s = 0.00460\,(1 + 1/1.76) = 0.00720$$

From equation (34) the neutral axis depth ratio for the section

$$n/d = 2 \times 26.2 \times 0.00720/1.50 = 0.251$$

This is less than the limiting neutral axis depth ratio for primary tension failure, $n_o/d = 0.318$.

From equation (15) the moment arm coefficient

$$\alpha = 1 - 0.39 \times 0.251 = 0.902$$

The ultimate resistance moment (from equation (36))

$$M_u = 157{,}000 \times 0.883 \times \left[0.902 \times 16 + (2 - 0.39 \times 0.251 \times 16)/1.76\right]$$
$$= 2{,}210{,}000\text{ lb. in.}$$

(*b*) *Ducts not bonded to concrete*

($K'_B = 0.25$ and $K'_T = 0.40$).

The greatest neutral axis depth ratio for primary tension failure (calculated in Example 1) $n_o/d = 0.114$. This is much lower than the value obtained in the previous paragraph, and the section is thus likely to show a primary compression failure. The neutral axis depth is therefore obtained from equation (46).

The ratios

$$r'_B + r'_T = 0.00920$$
$$K'_B r'_B + K'_T r'_T = (0.25 + 0.40) \times 0.00460 = 0.00299$$

and

$$K'_B r'_B + K'_T r'_T d_T/d = (0.25 + 0.05) \times 0.00460 = 0.00138$$

Substituting into equation (46)

$$(n/d)^2 + (2/1.50) \times (0.00299 \times 5.8 - 0.00920 \times 89{,}000/6{,}000)\, n/d = 2 \times 5.8 \times 0.00138/1.50$$

This gives the neutral axis depth ratio

$$n/d = 0.0798 + \sqrt{(0.0064 + 0.0106)} = 0.210$$

which is, in fact, greater than n_o/d.

The neutral axis depth $n = 3.36$ in., and the moment arm $d - \gamma n = 16 - 0.39 \times 3.36 = 14.69$ in.

The ultimate resistance moment for primary compression failure is given, as a first approximation, by equation (48)

$$M_u = \tfrac{1}{2} \times 6{,}000 \times 12 \times 3.36 \times 1.5 \times 14.69 - 89{,}000 \times 0.883\ (16\text{—}2)$$
$$= 2{,}660{,}000 - 1{,}100{,}000 = 1{,}560{,}000\text{ lb.}$$

A more accurate result is obtained from equation (4

$$M_u = 2{,}660{,}000 - \left[89{,}000 - 5.8 \times 0.4 \times 6{,}000 \times (3.36 - 2)/3.36\right] \times 0.883 \times 14 = 1{,}630{,}000\text{ lb. in.}$$

Example 3. Calculate the ultimate resistan moment of the concrete *I*-section shown in Fig. 1 which is post-tensioned by three cables each containi sixteen 0.2 in. diameter wires, (*a*) if the ducts a grouted after the completion of the tensioning proce and (*b*) if the wires are not bonded to the concrete.

The top flange has a width of $b = 22$ in. and a thic ness $D_T = 6$ in. The section is reinforced wi $A_s = 1.51$ sq. in. of steel at an effective depth $d = 40 - 3 = 37$ in. Therefore the ratios $r' = 1.5$ $22 \times 37 = 0.00185$; and $D_T/d = 6/37 = 0.162$.

It is assumed that the concrete has an ultimate cor pressive strength $\varnothing_{cu} = 5{,}400$ lb. per sq. in. (corr sponding approximately to a cube crushing strength 6,000 lb. per sq. in.) ; a modulus of elasticity $E_c = 4 \times 1$ lb. per sq. in. ; and an ultimate compressive stra $e_{cu} = 3 \times 10^{-3}$. The inelastic strain ratio is the $\beta = 0.55$, and the constant $\gamma = 0.40$.

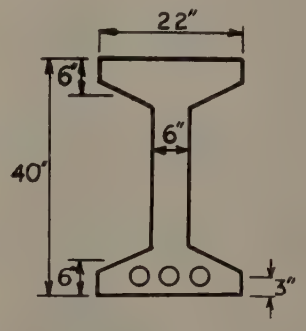


Fig. 10

It is assumed that the steel is initially stressed t 135,000 lb. per sq. in. Allowing 15 per cent. for lo due to creep etc., and taking the equivalent transf loss constant $\theta' = 0.95$, the equivalent external pr stress

$$f_{sp} = 0.85 \times 135{,}000/0.95 = 121{,}000\text{ lb. per sq. in}$$

Taking the modulus of elasticity as 28×10^6 lb. p sq. in., the corresponding strain $e_{sp} = 4.3 \times 10^{-3}$. is further assumed that the ultimate tensile strength the wire $\varnothing_{su} = 100$ tons per sq. in. = 224,000 lb. p sq. in., and its equivalent yield strain $e_{sy} = 10 \times 10^{-}$

The ultimate strength ratio $\rho = 224{,}000/5{,}400 = 41$.

It is assumed that the strain factor $K' = 0.95$ whe the cable ducts are grouted, and 0.25 when they are nc grouted.

(*a*) *Ducts grouted* ($K' = 0.95$).

The greatest neutral axis depth ratio for primary tension failure (from equation (10)).

$$n_o/d = 0.95 \times 3 \times 10^{-3} /$$
$$(0.95 \times 3 \times 10 - 4.3) \times 10^{-3} = 0.334$$

The neutral axis depth ratio for the section (from equation (14))

$$n/d = 2 \times 41.5 \times 0.00185 / 1.55 = 0.099$$

Since $n < n_o$, failure is initiated by yielding of the steel (primary tension failure). Since $n < D_T$, the neutral axis lies inside the top flange, and the section is therefore equivalent to a rectangular section.

From equation (16) the moment arm coefficient

$$\alpha = 1 - 0.40 \times 0.099 = 0.960$$

From equation (17) the ultimate resistance moment

$$M_u = 224{,}000 \times 1.51 \times 0.960 \times 37$$
$$= 12{,}020{,}000 \text{ lb. in.}$$

(*b*) *Wires not bonded to concrete.* ($K' = 0.25$.)

Since the area of steel is relatively small, the ultimate resistance moment remains unaltered if the cable ducts are not grouted. Taking the strain factor $K' = 0.25$, the greatest neutral axis depth ratio for primary tension failure

$$n_o / d = 0.25 \times 3 \times 10^{-3} /$$
$$(0.25 \times 3 + 10 - 4.3) \times 10^{-3} = 0.116$$

This is greater than 0.099, and the failure is therefore still caused by yielding of the steel. Since the strain factor does not enter into the calculations for the ultimate resistance moment for primary tension failure, $M_u = 12{,}020{,}000$ lb. in., as for the section with grouted ducts.

Example 4. Calculate the ultimate resistance moment of the section considered in Example 3, if the ultimate compressive strength of the concrete is 8,000 lb. per sq. in.

It is assumed that the ultimate compressive concrete strain remains $e_{cu} = 3 \times 10^{-3}$. The inelastic strain ratio is then $\beta = 0.33$, and the ratio $\gamma \times 0.36$. The ultimate strength ratio $\rho = 28$.

The neutral axis depth ratio

$$n / d = 2 \times 28 \times 0.00185 / 1.33 = 0.078$$

which is less than the limiting neutral axis depth ratio calculated in Example 3. Failure is therefore initiated by yielding of the steel, and the neutral axis lies inside the flange.

The moment arm coefficient $\alpha = 1 - 0.36 \times 0.078 = 0.972$, and the ultimate resistance moment

$$M_u = 224{,}000 \times 1.51 \times 0.972 \times 37$$
$$= 12{,}180{,}000 \text{ lb. in.}$$

This is only very little more than the moment calculated in Example 3, and the ultimate resistance moment is evidently not affected to any appreciable extent by a variation in the concrete strength if failure is initially caused by yielding of the steel. Any variation in the ultimate tensile strength of the steel, on the other hand, causes a proportional change in the ultimate moment.

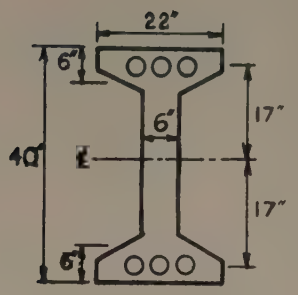


Fig. 11

Example 5. Calculate the ultimate resistance moment of the concrete *I*-section shown in Fig. 11 (which is uniformly post-tensioned by six cables each containing sixteen 0.2 in. diameter wires) if the ducts are grouted after completion of the tensioning process.

The properties of the materials are the same as in Example 3(*a*), viz. $f_{sp} = 121{,}000$ lb. per sq. in.; $\varnothing_{su} = 224{,}000$ lb. per sq. in.; $\rho = 41.5$; $\beta = 0.55$; $\gamma = 0.40$; and $n_o/d = 0.334$.

The ratio $\rho_s = 224{,}000 \times 121{,}000 = 1.85$

The dimensions of the concrete section are the same as in Example 3, and it is reinforced with $A_{sT} = A_{sB} = 1.51$ sq. in. of steel at the centre of each flange. Therefore the ratio $D_T/d = 0.162$, and the steel ratios for the top and bottom flanges respectively $r'_T = r'_B = 0.00185$. The equivalent steel ratio for the whole section

$$r'_B + r'_T/\rho_s = 0.00185 + 0.00100 = 0.00285$$

From equation (34) the neutral axis depth ratio

$$n/d = 2 \times 41.5 \times 0.00285 / 1.55 = 0.153$$

Since $n < D_T < n_o$, the neutral axis lies inside the top flange, and failure is caused initially by yielding of the steel.

The moment arm coefficient $\alpha = 1 - 0.40 \times 0.153 = 0.939$, and the ultimate resistance moment (from equation (35))

$$M_u = 224{,}000 \times 1.51 \times$$
$$\left[0.939 \times 37 + (3.0 - 0.40 \times 0.153 \times 37/185)\right]$$
$$= 224{,}000 \times 1.51 \times (34.8 + 0.4)$$
$$= 11{,}900{,}000 \text{ lb. in.}$$

From a comparison with the result obtained in Example 3 it is evident that the ultimate moment is reduced very slightly by the addition of the three cables in the top flange.

A simpler approximate solution is obtained from equation (36)

$$M_u = 224{,}000 \times 1.51 \times 0.939 \times 37$$
$$= 11{,}750{,}000 \text{ lb. in.}$$

Example 6. Calculate the ultimate resistance moment of the concrete *I*-section shown in Fig. 12, which is post-tensioned by a cable containing thirty-two 0.2 in. diameter wires and two cables each containing twenty-four wires of the same diameter. The centres of the cables are placed at a distance of 3 in. from the bottom face of the section, and the cable ducts are grouted after completion of the tensioning process.

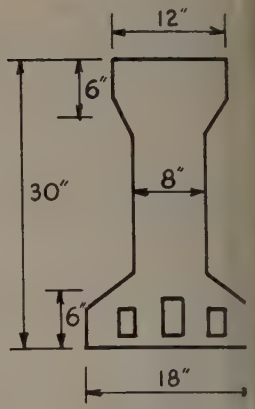


Fig. 12

The properties of the concrete and the steel are the same as in Example 3(*a*), viz. $\varnothing_{cu} = 5{,}400$ lb. per sq. in.; $E_c = 4 \times 10^6$ lb. per sq. in.; $\beta = 0.55$; $\gamma = 0.40$; $f_{sp} = 121{,}000$ lb. per sq. in.; $\varnothing_{su} = 224{,}000$lb. per sq. in.; $e_{sp} = 4.3 \times 10^{-3}$; $e_{sy} = 10 \times 10^{-3}$; $\rho = 41.5$; $K' = 0.95$; and $n_o/d = 0.334$.

The top flange had a width of $b = 12$ in., and thickness $D_T = 6$ in. The thickness of the rib $b = 8$ in. The section is reinforced with $A_s = 2.514$ sq. in. of steel at an effective depth $d = 30 - 3 = 27$ in. Therefore the ratios $r' = 2.514 / 12 \times 27 = 0.00776$ and $D_T/d = 0.222$.

The neutral axis lies outside the flange if the steel ratio exceeds (from equation (14))

$$r' = \tfrac{1}{2}(1 + \beta)\, D_T/d\rho = \tfrac{1}{2} \times 1.55 \times 0.222 / 41.5 = 0.00414$$

and the neutral axis is therefore in the rib.

(*a*) *Neglecting the compression in the concrete rib.*

From equation (27) the ultimate resistance moment

$$M_u = 5{,}400 \times 12 \times 6\,(27 - \tfrac{1}{2} \times 6) = 9{,}340{,}000 \text{ lb. in.}$$

(*b*) *Allowing for the compression in the concrete rib.*

A more accurate assessment of the ultimate resistance moment of the section is obtained if the compressiv stresses in the concrete rib are included. The position of the neutral axis must then be determined, and th strain in the steel related to the strain at the top of the compression flange. Using the simplified stress-stra diagram for steel shown in Fig. 2, the effective modulus of elasticity

$$E'_s = (\varnothing_{su} - f_{sp}) / (e_{sy} - e_{sp}) = (224{,}000 - 121{,}000) / (10 - 4.3) \times 10^{-3} = 18 \times 10^6 \text{ lb. per sq. in.}$$

The effective modular ratio is therefore

$$m' = E'_s / E_c\,(1 - \beta) = 18 / 4 \times 0.45 = 10.0$$

Substituting into equation (29)

$$(n/d)^2 + (2/1.55) \times \left[(12/8) \times (10 \times 0.00776 \times 0.95 - 0.00776 \times 121{,}000/5{,}400 + 6/27) - 6/27\right] n/d$$
$$= 2 \times 10 \times 0.95 \times 0.00776 \times 12/8 \times 1.55$$

Evaluating the principal terms

$$(n/d)^2 + 1.29 \left[1.5\,(0.0737 - 0.1740 + 0.2222) - 0.2222\right] \quad n/d = 0.1428$$

which gives the neutral axis depth ratio

$$n/d = 0.254 + \sqrt{(0.0016 + 0.1428)} = 0.404$$

Since $n > n_o$, failure is caused by crushing of the concrete.

The neutral axis depth $n = 0.404 \times 27 = 10.91$ in.,

and the moment arm for the rib $d - \gamma n = 27 - 0.40 \times 10.91 = 22.64$ in.

Substituting into equation (30), the ultimate resistance moment

$$M_u = 5{,}400 \left[\tfrac{1}{2} \times 8 \times 10.91 \times 1.55 \times 22.64 + (12 - 8) \times 6 \times (27 - \tfrac{1}{2} \times 6)\right] = 5{,}400\,(1.532 + 576) = 11{,}380{,}000 \text{ lb. in.}$$

Due to the small width of the top flange relative to the rib, the error resulting from the neglect of the compression in the concrete rib is evidently appreciable.

Example 7. Calculate the ultimate resistance moment of the *I*-section shown in Fig. 13, which is pre-tensioned by 20 No. 12 S.W.G. wires placed symmetrically about the centre of the bottom flange.

The top flange has a width $b = 10$ in. and a thickness $D_T = 2$ in. The section is reinforced with $A_s = 0.170$ sq. in. of steel at an effective depth $d = 10 - 1 = 9$ in. Therefore the ratios $r' = 0.170/10 \times 9 = 0.00189$, and $D_T/d = 0.222$.

The concrete is assumed to have the same properties as in Example 3, viz. $\varnothing_{cu} = 5{,}400$ lb. per sq. in.; $e_{cu} = 3 \times 10^{-3}$; $\beta = 0.55$; and $\gamma = 0.40$.

It is assumed that the steel is initially stressed to 60,000 lb. per sq. in. Allowing 15 per cent. for loss on transfer and 10 per cent. for loss after transfer, and taking the "equivalent transfer loss" constant $\theta' = 0.95$, the equivalent external prestress

$$f_{sp} = 0.85 \times 0.90 \times 160{,}000 / 0.95$$
$$= 129{,}000 \text{ lb. per sq. in.}$$

Taking the modulus of elasticity as 28×10^6 lb. per sq. in., the corresponding strain $e_{sp} = 4.6 \times 10^{-3}$. It is further assumed that the ultimate tensile strength of the wire $\varnothing_{su} = 120$ tons per sq. in. $= 268{,}000$ lb. per sq. in., and its equivalent yield strain $e_{sy} = 10 \times 10^{-3}$.

The ultimate strength ratio $\rho = 268{,}000/5{,}400 = 49.6$.

It is assumed that the strain factor $K' = 1$ for pre-tensioned wires with a suitably rough surface.

The greatest neutral axis depth ratio for primary tension failure (from equation (10))

$$n_o / d = 3 \times 10^{-3} / (3 + 10 - 4.6) \times 10^{-3} = 0.357.$$

The neutral axis depth ratio for the section

$$n/d = 2 \times 49.6 \times 0.00189 / 1.55 = 0.121$$

Since $n < D_T < n_o$, the neutral axis lies inside the top flange, and failure is caused initially by yielding of the steel.

The moment arm coefficient $\alpha = 1 - 0.40 \times 0.121 = 0.952$, and the ultimate resistance moment

$$M_u = 268{,}000 \times 0.170 \times 0.952 \times 9$$
$$= 391{,}000 \text{ lb. in.}$$

Conclusion

The application of the theory set out in the preceding pages requires a knowledge of some of the properties of the steel and the concrete. The ultimate strength of the steel and the concrete are easily determined. The limiting strains and the strain factor, however, are dependent on a number of variables, and are not therefore readily ascertained at the design stage; it is generally necessary to assume average values for them.

The ultimate strength for primary tension failure can be determined quite accurately, since it depends almost entirely on the ultimate strength of the steel. In computing the ultimate strength for primary compression failure, however, the strain factor K' (depending mainly on the bond between the steel and the concrete, and on the shape of the bending moment diagram), and the ultimate compressive strain of the concrete are required in most problems in addition to the concrete crushing strength; the same accuracy cannot therefore be expected.

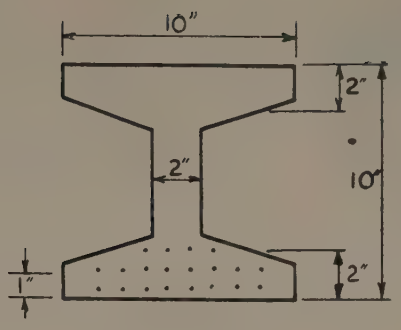


Fig. 13

There is also an appreciable difference between the complexity of the formulae for primary tension and compression failure. The calculations for a primary tension failure are very simple. This is not generally the case for a primary compression failure.

It is therefore fortunate that in practice very few prestressed beams show primary compression failure. In the majority of cases the assessment of the load factor against failure thus presents little difficulty.

Provided the neutral axis depth is less than the upper limiting depth for primary tension failure, the ultimate resistance moment of a section with bottom steel only

$$M_u = \varnothing_{su} A_s \cdot \alpha d \quad . \quad . \quad . \quad . \quad . \quad (17)$$

where $\varnothing_{su}$ is ultimate tensile strength of the steel, A_s is its cross-sectional area, d is the effective depth, measured to the centre of the steel, and α is the moment arm coefficient. The numerical value of α depends mainly on the ratio (ρ) of the ultimate strength of the steel to that of the concrete, and on the steel ratio $r' = A_s / b\,d$ (where b is the width of the *top* flange). It depends to a lesser extent on the inelastic strain ratio (β) of the concrete. The chart in Fig. 5 is compiled for an average value of β, and should be sufficiently accurate for most purposes.

For a section with top and bottom steel

$$M_u = \varnothing_{su} A_{sB} \alpha d \quad . \quad . \quad . \quad . \quad . \quad (36)$$

where A_{sB} is the cross-sectional area of the bottom steel. The numerical value of α can be obtained from Fig. 5, using the equivalent steel ratio

$$r' = (A_{sB} + A_{sT} / \rho_s) / b\,d$$

where A_{sT} is the cross-sectional area of the top steel, and ρ_s is the ratio of the ultimate steel stress to the effective prestress, approximately equal to 2 in most cases.

When the neutral axis depth is greater than the upper limiting depth for primary tension failure, the ultimate resistance moment cannot be obtained with the same degree of accuracy. A simple approximate solution for flanged sections is obtained by neglecting the stresses in the concrete rib. This generally errs on the safe side, but is applicable only when the neutral axis lies outside the flange. The ultimate resistance moment for a flanged section with bottom steel only

$$M_u = \varnothing_{cu} b\, D_T (d - \tfrac{1}{2} D_T) \quad . \quad . \quad . \quad (27)$$

where $\varnothing_{cu}$ is the ultimate compressive strength of the concrete (approximately equal to 90 per cent. of the cube crushing strength) and D_T is the thickness of the flange.

The ultimate resistance moment of a flanged section with top and bottom steel

$$M_u = \varnothing_{cu} b D_T (d - \tfrac{1}{2} D_T) - f_{sp} A_{sT} (d - d_T) \quad (52)$$

where d_T is the depth of the centre of the top steel from the compression face, and f_{sp} is the equivalent external prestress (which is generally about 5 per cent. greater than the final prestress remaining after allowance for all losses).

The remaining solutions for primary compression failure require a knowledge of the ultimate compressive strain of the concrete and of the strain factor, and the equations are more complicated. For all but the highest strength concrete an ultimate strain of 0.3 per cent. may be considered a conservative value. The strain factor generally varies between 0.85 and 1.0 for properly bonded or grouted beams. When the steel is not bonded to the concrete, however, the strain factor is much lower, and the ultimate strength may be greatly reduced in consequence.

Appendix

a	length of arm of ultimate resistance moment.
A_c	cross-sectional area of homogeneous concrete section.
A_s, A_{sT}, A_{sB}	cross-sectional area of steel, of steel in top layer, of steel in bottom layer.
b	overall width of section.
b_r	width of rib.
C	compressive force.
d	effective depth of section, measured to centre of bottom steel.
d_T	depth of centre of top steel.
D	overall depth of section.
D_T	thickness of top flange.
e_c, e_{cT}	concrete strain in general, at top face of beam.
e_s, e_{sT}, e_{sB}	steel strain or change of steel strain in general, in top steel, in bottom steel.
e_{cu}, e_{su}	ultimate strain of concrete, of steel.
e_{cy}	equivalent yield strain of concrete.
e_{sp}	steel strain due to prestressing force P_e.
e_{sy}	lower limiting strain of the fully plastic range of the st
E_c, E_s	modulus of elasticity of concrete, steel.
E'_s	effective modulus of elasticity of steel for $e_{sp} < e_s < e$
f_c, f_s	stress in concrete, steel.
f_{sp}	steel stress due to equivalent external prestressing fo P_e.
K	strain factor (ratio of steel strain to strain in surround concrete) for homogeneous section.
K'	strain factor for cracked section.
m	modular ratio for homogeneous section $= E_s/E_c$.
m'	modular ratio for cracked section $= E'_s/E_c\ (1 - \beta)$
M_D, M_L	bending moment due to actual dead load, live load.
M_u	ultimate resistance moment of section.
n	depth of neutral axis.
n_r	limiting neutral axis depth for steel rupture.
n_o	steel ratio for simultaneous tension and compress failure.
P_e	equivalent external prestressing force, which wo leave concrete unstressed.
P_{eT}, P_{eB}	equivalent external prestressing force corresponding top, bottom steel.
P_f	final prestressing force after allowing for all losses.
r	steel ratio for homogeneous section, $= A_s/A_c$.
r'	steel ratio for cracked section, $= A_s/bd$.
r'_T, r'_B	top, bottom steel ratio for cracked section.
r'_r	limiting steel ratio for steel rupture.
r'_o	steel ratio for simultaneous tension and compressi failure.
T	tensile force.
α	moment arm coefficient $= a/d$.
β	inelastic strain ratio for concrete $= (e_{cu} - e_{cy})/e_{cu}$.
γ	$= (1 + \beta + \beta^2)\ /\ 3\ (1 + \beta)$.
θ'	equivalent transfer loss constant $= P_f/P_e$.
$\varnothing_{cu}$	ultimate strength in compression of concrete in p stressed beam.
$\varnothing_{c6}$	crushing strength of 6 in. concrete cube.
$\varnothing_{su}$	ultimate strength of steel.
ρ	ultimate stress ratio, $= \varnothing_{su}/\varnothing_{cu}$.
ρ_s	ratio of ultimate steel stress to prestress, $= \varnothing_{su}/f_{sp}$.

Book Reviews

Stress Concentration Design Factors, by R. E. Peterson. (New York : Wiley ; London : Chapman & Hall, 1953.) 155 pp., 11 in. × 9 in. Price 68s.

Following upon the failures of various constructions, in recent years the attention of engineers is being markedly intensified in the field of research in an endeavour to assess rationally all the contributory causes of fracture.

The interdependence of metallurgy and engineering is generally recognised and there is generally a demand for improved quality in materials. Whilst recognising this the author considers that failures are seldom due essentially to faulty material, but rather to imperfection in the design, with particular emphasis on the design of details.

In pursuing this theme the author proceeds to develop with mathematical and photoelastic illustrations the vital question of the stress concentration factor for a variety of details common to all branches of engineering, ranging from a simple normal test piece to the effect of grooves, holes, fillets, keyways, gear teeth, helical springs and rotating discs subjected to tension, bending and torsion, proceeding on to desi relations for steady stress, alternating stress and co bination of the two. The influence of various geomet features is well illustrated.

The author in propounding his mathematics of t subject has gone to great lengths to correlate all t results of research available, and in his acknowled ments and 174 references has given ample opportuni and guidance for further exploration for those who desire. The time for such research is not always ava able to the designer, however, and the author realisi this has commendably included 93 large size grap which permit speedy interpolation.

An attempt has been made to provide the desig with " reasonable " design information on notch sen tivity based on knowledge to date. This is appreciat as the ultimate solution of this phenomenon may be part metallurgical.

Whilst some designers may have seen " through glass but darkly," this publication will help to bring t problem of stress concentration into clearer relief.

G. S. G.

(*continued on page 222*)

Plastic Analysis and Design of Mild Steel Vierendeel Girders

by Arnold W. Hendry, D.Sc., Ph.D., A.M.Inst.C.E., M.I.Struct.E.

Summary

This paper discusses the application of the simple plastic theory to the design of mild steel Vierendeel girders. Three modes of failure are described and equations are derived for calculating the collapse load of a girder in each of these modes. Experimental results are quoted in confirmation and the application of the analysis to the design of parallel and inclined chord girders is illustrated by calculations for the design of a five panel bridge girder.

Introductory

The Vierendeel girder has not been widely adopted for bridge or building frame construction in Britain, no doubt because it is inherently less efficient and therefore heavier than a corresponding lattice girder. Nevertheless, there are occasions when a Vierendeel girder might be used in spite of this disadvantage, for example in a building frame where the absence of diagonals may be of great convenience in accommodating doors or windows or in bridgework where its clean appearance might lead to its adoption were it not for the rather laborious calculations which are necessary for the design of a girder of this type by elastic analysis. The work can certainly be simplified by the use of formulae such as those of Baes[1] but it remains difficult to visualise the behaviour of the structure and therefore to arrive at the most advantageous proportions.

The simple plastic theory which has already been applied to the design of a variety of mild steel structures offers a convenient method of analysing and designing Vierendeel girders of any outline. As will be demonstrated in this paper, the calculations are simple and a very clear impression is obtained of the behaviour of the structure.

Plastic Analysis of the Vierendeel Girder

In the analysis which follows it is assumed that failure of the structure takes place when a sufficient number of plastic hinges have been formed to permit large deflections. In simple structures the number of plastic hinges at failure is usually one more than the degree of redundancy of the structure, i.e. when the structure as a whole has become a mechanism. In a Vierendeel girder on the other hand failure of the structure as a whole invariably results from the local failure of a small number of its members and the number of plastic hinges at collapse is less than the degree of redundancy plus one. Typical plastic hinge locations in a Vierendeel girder at collapse are shown in Fig. 1. Three modes of failure can be distinguished :

A. By shear of a panel, in which case plastic hinges are formed at the ends of the chord members ;

B. By failure of the verticals, in which case plastic hinges are formed at the ends of a number of the verticals and at one section in each chord ;

C. By formation of plastic hinges in one or more of the verticals and at two sections in each chord.

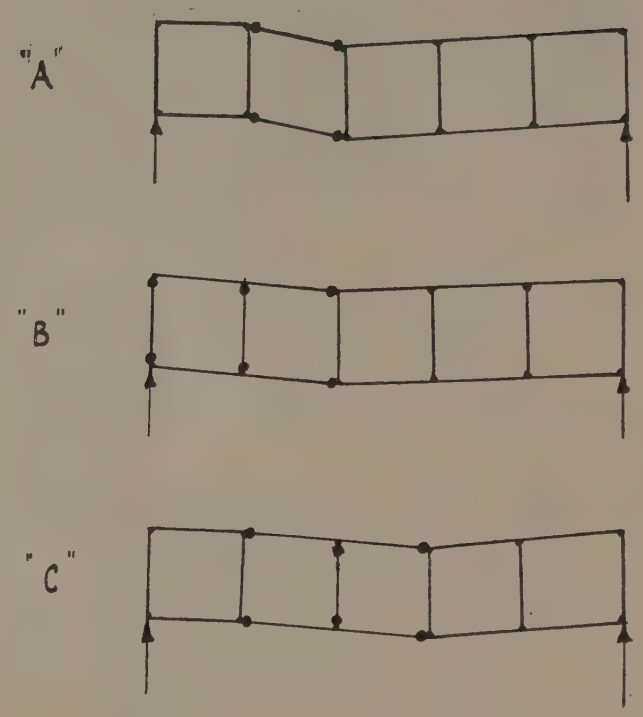


Fig. 1.—A, B, C. Modes of failure

The diagrams of Fig. 1 show parallel chord girders but they apply equally to inclined chord girders. The three modes of failure can be clearly distinguished in the photograph of actual test frames in Fig. 2.

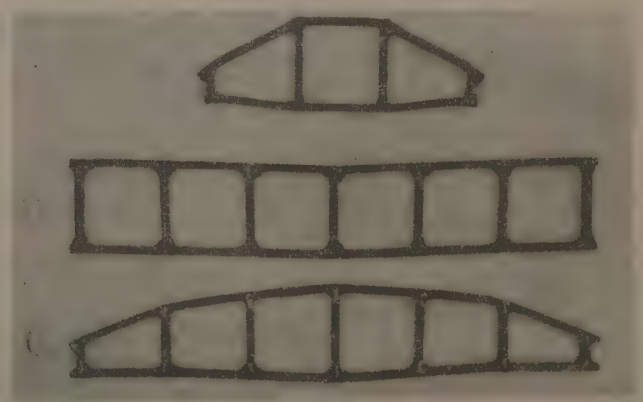


Fig. 2.—A, B, C. Test frames showing modes of failure

The fact that in practical circumstances failure of the truss will take place according to one or other of these modes makes it possible to write down equations from which the collapse load of a given structure can be calculated or the sections required to withstand a given loading can be determined. We may now, therefore, examine each of the above modes of failure in detail.

Fig. 3 shows the dimensions of a typical Vierendeel girder with inclined chords and Fig. 4 shows the failure of a panel according to mode A. At collapse there will be plastic moments at the ends of the chord members and points of inflection at their mid-points. Thus, considering half of the panel, the vertical and horizontal forces in the chords at the points of inflection will be as shown in Fig. 4(b) where S_x and M_x are the shear and bending moment on the truss at a section through the centre of the panel; γ is the ratio of the plastic moments of the top and bottom chords, i.e. m_p'/m_p''. Resolving along and at right angles to the axes of the members gives the forces shown in Fig. 4(c) and the collapse load is then obtained from the following :

$$S' \cdot \frac{p}{2\cos\alpha'} + S'' \cdot \frac{p}{2\cos\alpha''} = \left(m_p' + m_p''\right)$$

$$\left(\frac{\gamma}{\gamma+1} \cdot S_x \cos\alpha' - \frac{{}_A M_x}{h_x} \cdot \sin\alpha'\right)\frac{p}{2\cos\alpha'}$$

$$+\left(\frac{S_x}{\gamma+1}\cos\alpha'' - \frac{{}_A M_x}{h}\sin\alpha''\right)\frac{p}{2\cos\alpha''} = m_p' + m_p''$$

$$\text{i.e. } \frac{p}{2}\left[S_x - \frac{{}_A M_x}{h_x}(\tan\alpha' + \tan\alpha'')\right]$$
$$= (m_p' + m_p'') \quad . \quad . \quad . \quad (1)$$

If the axial load in the members is greater than abo 2.5 tons/sq. in., m_p' and m_p'' will be appreciably low than the plastic moments in pure bending. This effe has been fully described elsewhere[2, 3] and a reducti table for a yield stress of 15.25 tons/sq. in. is given an appendix to this paper for convenience of referen

It is convenient to express h_x and $\tan\alpha$ in equati (1) in terms of the heights of the verticals, thus :

$$\frac{p}{2}\left(S_x - 2 \cdot \frac{{}_A M_x}{p} \cdot \frac{h_{n+1} - h_n}{h_{n+1} + h_n}\right)$$
$$= m_p' + m_p'' \quad . \quad . \quad . \quad (1$$

with the notation of Fig. 3.

It will be observed that the sections of the vertic do not enter into equation (1) but obviously the members must be capable of transmitting the verti loads between the chords and at the same time resisting the horizontal shear in the girder. If t verticals are insufficient to permit the development the shear strength of the panels, i.e. failure by mode then failure will take place according to mode B mode C. Referring to mode B, Figs. 5(a), (b) and show sections through the points of inflection of t verticals and the top chord member in the first, seco and third panels of a truss respectively. If F_1, F_2 a F_3 are the horizontal components of the loads in t chord members and H_1, H_2 and H_3 are the maximu horizontal loads which can be developed at the mi points of the verticals, it is clear that :

$$F_1 \leq H_1$$
$$F_2 \leq H_1 + H_2$$
$$F_3 \leq H_1 + H_2 + H_3$$

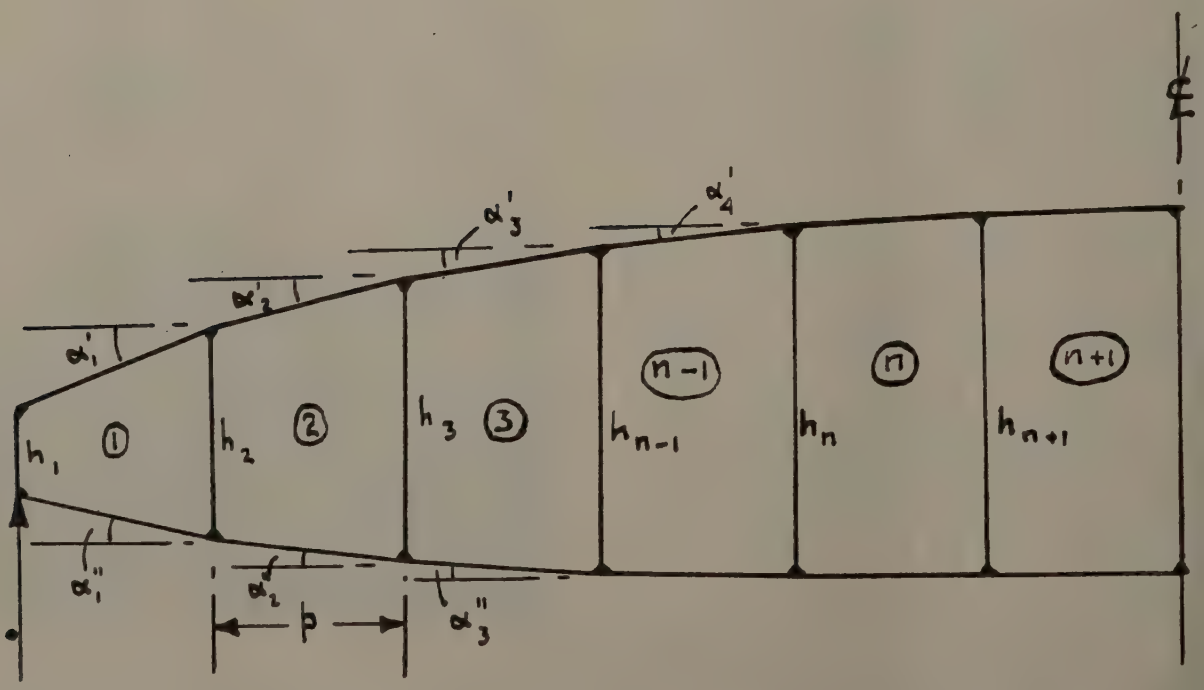


Fig. 3.—Dimensions of girder

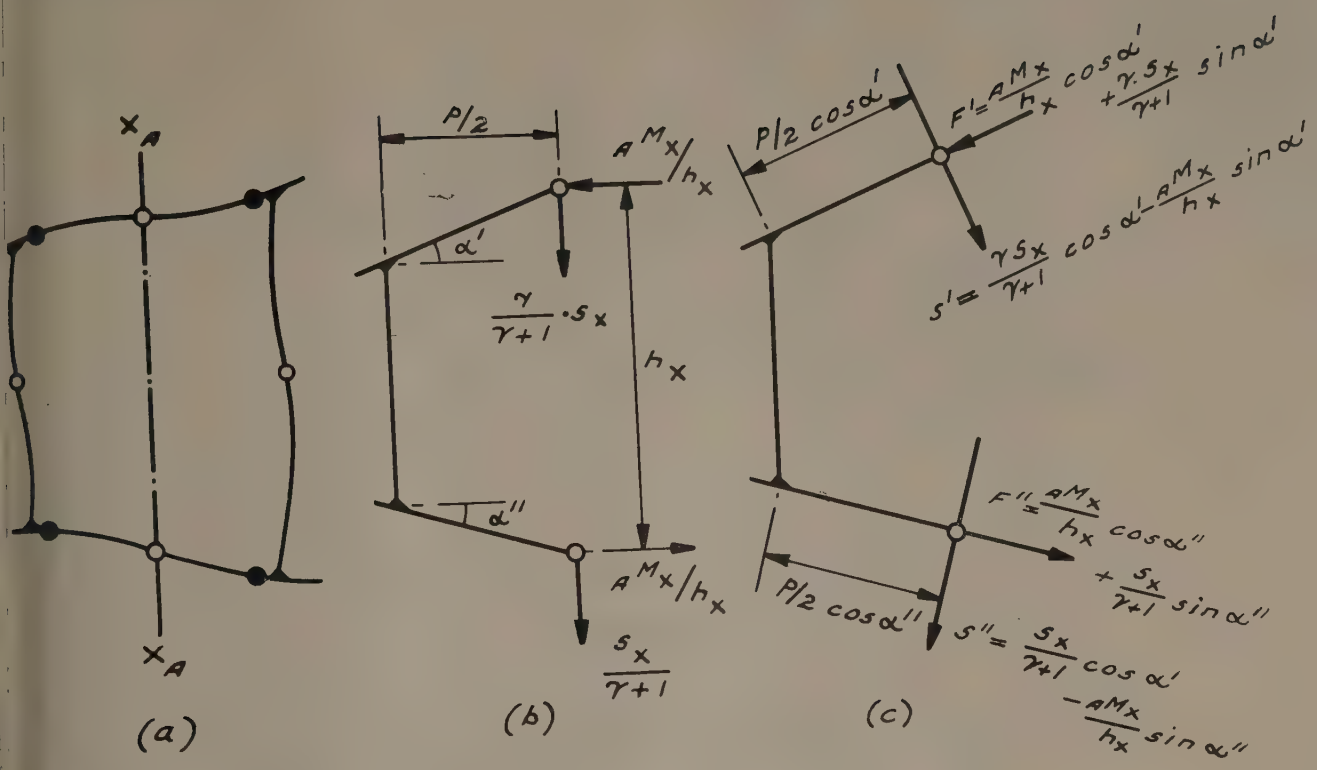


Fig. 4.—Failure by mode "A"

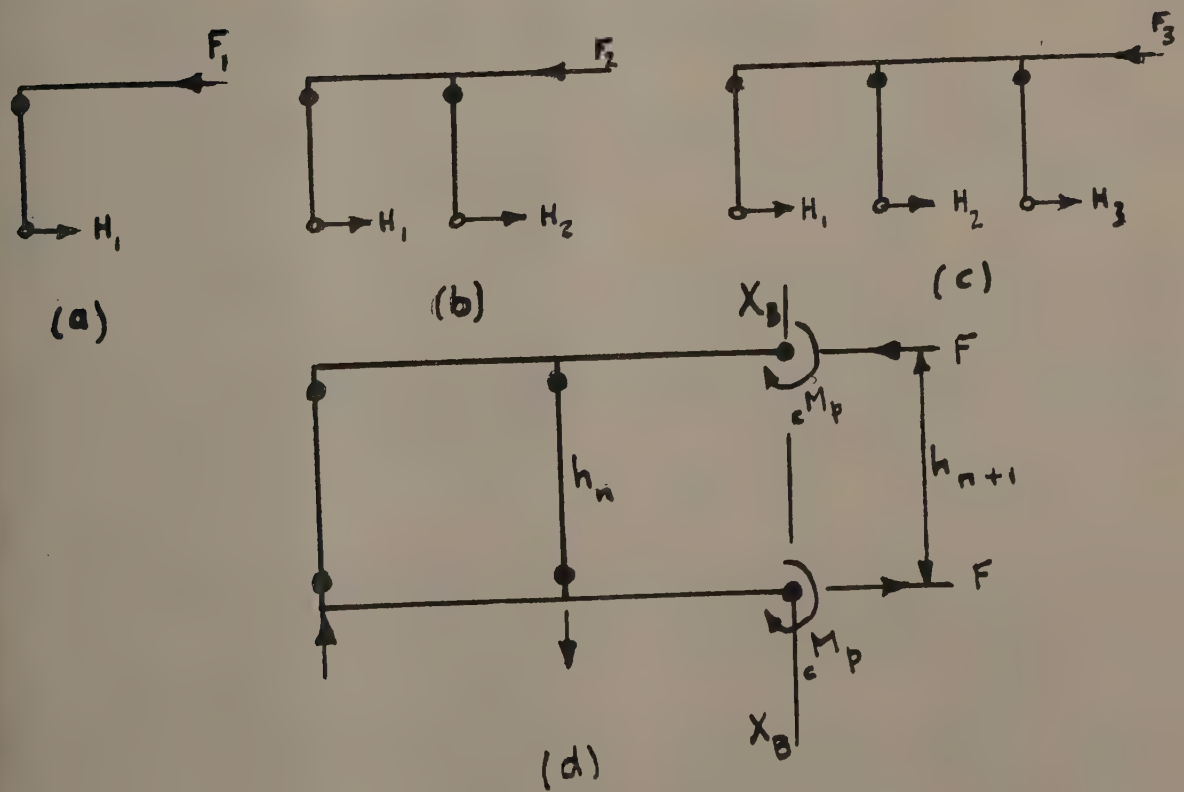


Fig. 5.—Failure by mode "B"

It follows that the plastic moments of the verticals must be such as to permit the development of sufficiently large horizontal forces in the verticals to meet these requirements since the maximum horizontal force which can be developed by a vertical is limited by the formation of a plastic hinge at its junction with the chord, as indicated in Fig. 5. This type of failure will not take place until the deflection of the truss as a whole is large and this entails the formation of plastic hinges in the chord members as well, as indicated in Fig. 1B; the positions of these plastic hinges depend of course on the loading and frame dimensions. The

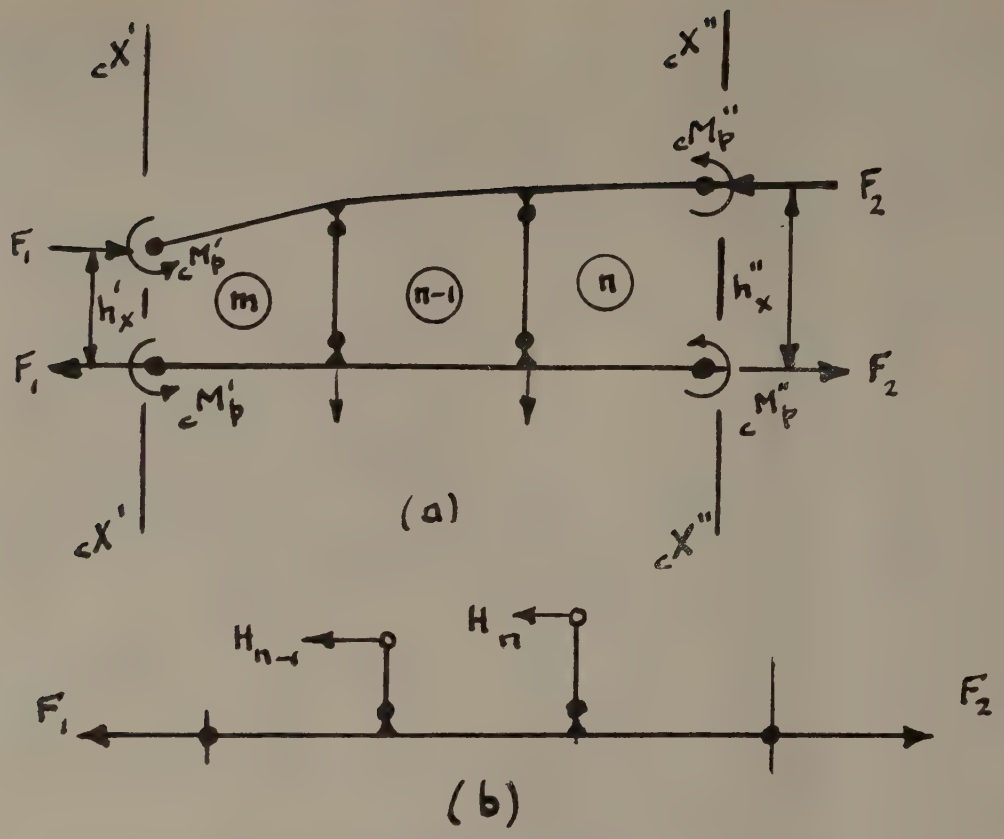


Fig. 6.—Failure by mode "C"

collapse load for a girder failing in this way can be calculated from the expression :

$$_{B}M_{x} = F \cdot h_{n+1} + 2_{c}M_{p}$$

where $_{B}M_{x}$ is the bending moment at section X_B in the truss (Fig. 5(c)), F is the maximum horizontal component of the chord load which can be resisted by the verticals up to section X_B, h_{n+1} is the height of the $(n + 1)$th vertical and $_{c}M_{p}$ is the plastic moment of the chord members. $_{B}M_{x}$ can be expressed in terms of the collapse load and F is given by :

$$F = \sum_{1}^{n} \frac{2 \cdot {}_{v}M_{p}}{h}$$

in which $_{v}M_{p}$ and h are respectively the plastic moment and height of a vertical. Thus for failure by mode B :

$$_{B}M_{x} = h_{n+1} \cdot \sum_{1}^{n} \frac{2 \cdot {}_{v}M_{p}}{h} + 2_{c}M_{p} \quad . \quad . \quad (2)$$

Inspection of Fig. 1 will show that failure by mode C is similar to mode B except that the plastic hinges nearest the left hand support are formed in two of the chord members instead of in a vertical. Considering the portion of the structure shown in Fig. 6(a) it will be seen that :

$$F_1 = \frac{{}_{c}M_{x}' + 2_{c}M_{p}'}{h_{x}'}$$

$$\text{and } F_2 = \frac{{}_{c}M_{x}'' - 2 \cdot {}_{c}M_{p}''}{h_{x}''}$$

From Fig. 6(c) :

$$F_2 - F_1 = \sum_{m}^{n} \frac{2 \cdot {}_{v}M_{p}}{h}$$

$$\left(\frac{{}_{c}M_{x}'' - 2\,{}_{c}M_{p}''}{h_{x}''}\right) - \left(\frac{{}_{c}M_{x}' + 2 \cdot {}_{c}M_{p}'}{h_{x}'}\right)$$

$$= \sum_{m}^{n} 2 \cdot \frac{{}_{v}M_{p}}{h} \quad . \quad . \quad . \quad (3$$

As in the previous case the moments $_{c}M_{x}'$ and $_{c}M_{x}$ can be evaluated in terms of the collapse load.

Experiments on the Strength of Vierendeel Girders

A number of experiments have been carried out t provide confirmation of the plastic analysis of the fore going section.

Group I : Parallel chord girders. This group com prised five frames having 4" square panels and member of rectangular section. The frames were welded up i a jig from $\frac{3}{4}" \times \frac{3}{8}"$ flat sections—bending of the mem bers in the structure taking place about the minor axi of the rectangular section. Details of the frames an load positions are given in Fig. 7 ; the tests were carrie out in an Avery hydraulic testing machine deflectio measurements having been made by dial gauges.

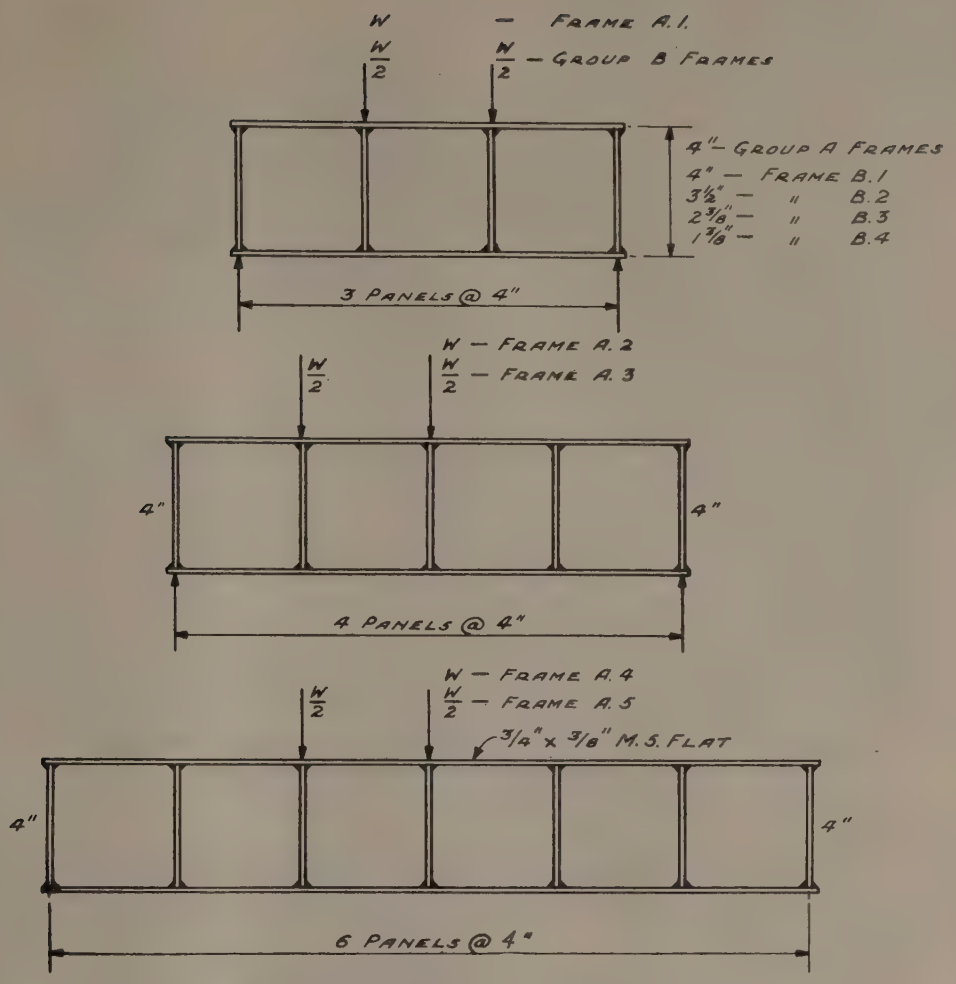


Fig. 7.—Dimensions of Frames and Loading

A preliminary test on a piece of the $\frac{3}{4}'' \times \frac{3}{8}''$ flat from which the models were made up gave a value of M_p of 0.585 in. ton, so that the collapse load of Frame No. 1.1 for failure by modes A, B, and C is found as follows :

Mode A—Failure of first panel.

In equation (1) :

Effective panel length (allowing for welds) $= (4.0 - 0.375 - 0.25)$
$= 3.365$ ins.

$$S_x = \frac{2W_c}{3} \qquad h_{n+1} = h_n$$

$$m_p' = m_p'' = 0.585 \text{ in. ton}$$

and

$$W_c = \frac{2 \times 0.585 \times 3}{3.365} = 1.04 \text{ ton.}$$

Mode B—Failure of first panel.

$$_BM_x = \frac{2W_c}{3} \times 3.712 = \frac{7.424}{3} W_c$$

$$h_{n+1} = 4_{in.} \qquad \sum_1^n \frac{2_v M_p}{h} = \frac{2 \times 0.585}{3.365}$$

$$\therefore \frac{7.424}{3} W_c = \frac{4 \times 2 \times 0.585}{3.365} + 2 \times 0.585$$

and

$$W_c = \frac{3 \times 4 \times 2 \times 0.585}{7.424 \times 3.365} + \frac{3 \times 2 \times 0.585}{7.424}$$

$$= 1.04 \text{ ton.}$$

Failure of second and third panels.

$$_BM_x = \frac{W_c}{3} \times 7.712 = \frac{7.712}{3} W_c$$

$$h_{n+1} = 4^{in.} \qquad \sum_1^n \frac{2_v M_p}{h} = \frac{2 \times 2 \times 0.585}{3.365}$$

$$\therefore \frac{7.712}{3} \cdot W_c = \frac{4 \times 2 \times 2 \times 0.585}{3.365} + 2 \times 0.585$$

and

$$W_c = \frac{3 \times 4 \times 2 \times 2 \times 0.585}{7.712 \times 3.365} + \frac{3 \times 2 \times 0.585}{7.712}$$

$$= 1.65 \text{ ton.}$$

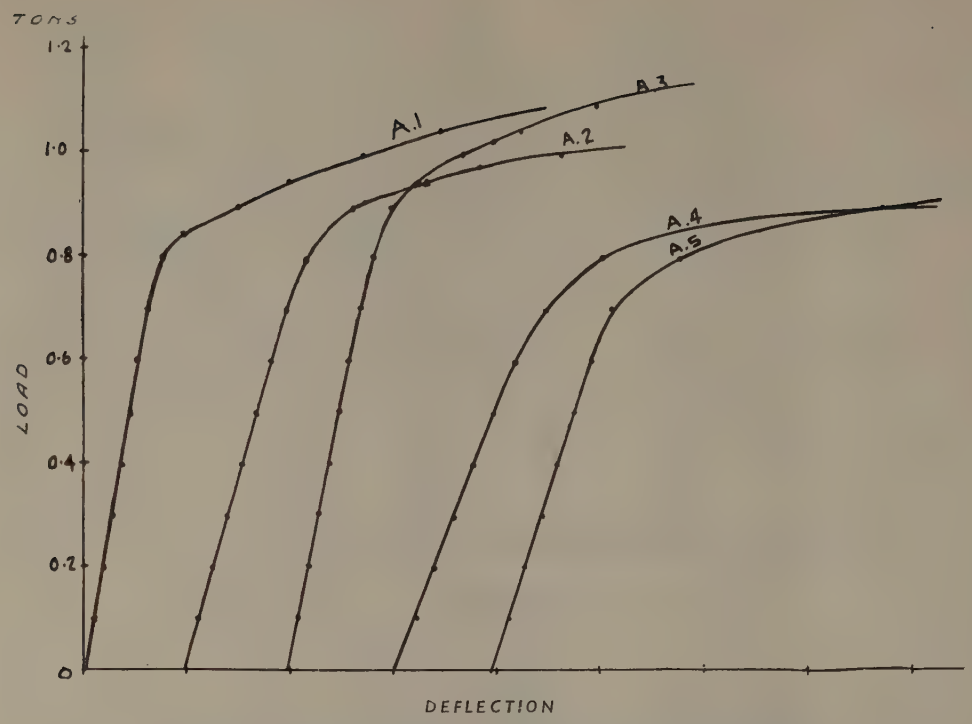


Fig. 8.—Load deflection curves for Group I girders

Mode C—Failure of second and third panels.

$$_cM_x' = 0 \quad _cM_x'' = \frac{1}{3} W_c \times 7.712$$

$$h_x' = h_x'' = 4.0 \text{ ins.}$$

$$\therefore \tfrac{1}{3} \frac{W_c \times 7.712 - 2 \times 0.585}{4.0} = \frac{0 + 2 \times 0.585}{4.0}$$

$$= \frac{2 \times 2 \times 0.585}{3.365}$$

and

$$W_c = 0.585 \left(\tfrac{1}{2} + \tfrac{1}{2} + \frac{4.00}{3.36} \right) \frac{12.0}{7.712} = 1.98 \text{ tons}$$

Failure of the truss will therefore take place at 1.04 ton by modes A and B simultaneously ; this means that plastic hinges will be formed at the end of the first chord members and also at the ends of the first vertical. According to the test result the frame failed at 1.08 tons. The shape of the load—deflection diagram suggests that the higher value of the experimental result might be accounted for by strain hardening.

Similar results were obtained for the other frames of this group and are given in Table I, while the load-deflection curves for these frames are shown in Fig. 8.

Table I

Frame No.	Collapse Load (tons)			
	Experimental	Calculated		
		A	B	C
1.1	1.08	**1.04**	**1.04**	1.98
1.2	1.02	1.39	0.99	**0.96**
1.3	1.14	**1.12**	1.48	1.29
1.4	0.89	1.39	**0.88**	**0.88**
1.5	0.93	1.19	0.85	**0.83**

To indicate the mode of failure in Table I the min mum calculated collapse load is shown in heavy typ The experimental and calculated results are in fa agreement ; the experimental result is consistentl higher than the calculated one probably as the resu of strain hardening. The appearance of the frames afte test confirmed the type of failure.

Group II : Inclined Chord Frames. The frames i this group each had three panels the central pan being 4″ square and the height of the end vertica varying between 1—$\frac{3}{8}$″ and 4″. The frames wer loaded were equal loads at the interior panel poin and failure took place in each case by mode A. A com parison between calculated and experimental results shown in Fig. 9, and within the limits of the tests, goo agreement is obtained. The steep rise in the strengt of a panel against failure by mode A as the inclinatio of the chords is increased is clearly demonstrated.

Group III : Inclined Chord Frames. To provid further confirmation of the behaviour of inclined chor frames, two more such frames were tested. Each ha six panels, a straight lower chord and vertical height as indicated in Fig. 10. Frame No. 3.1 carried a centra load while No. 3.2 carried equal loads at the third an fourth panel points. The collapse loads for the thre modes of failure are given in Table II.

Table II

Frame No.	Collapse Load (tons)			
	Experimental	Calculated		
		A	B	C
3.1	1.47	1.99	1.62	1.43
3.2	1.36	1.44	1.40	1.32

It will be appreciated that a frame may fail by given mode in one of several ways (e.g. any one pane

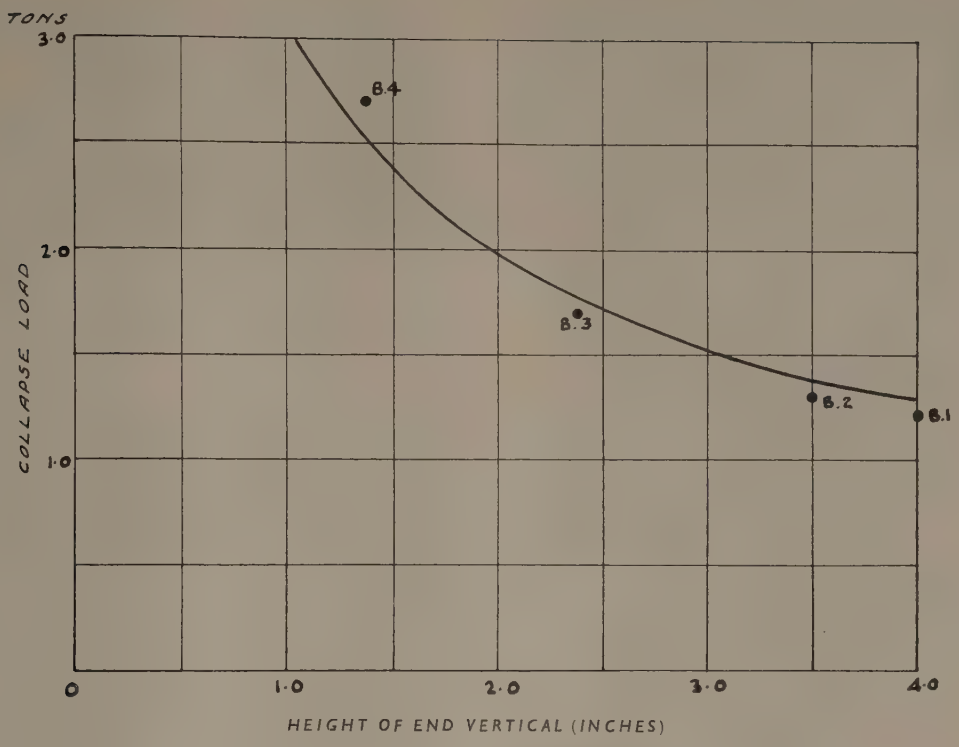


Fig. 9.—Calculated and experimental results for Group II girders

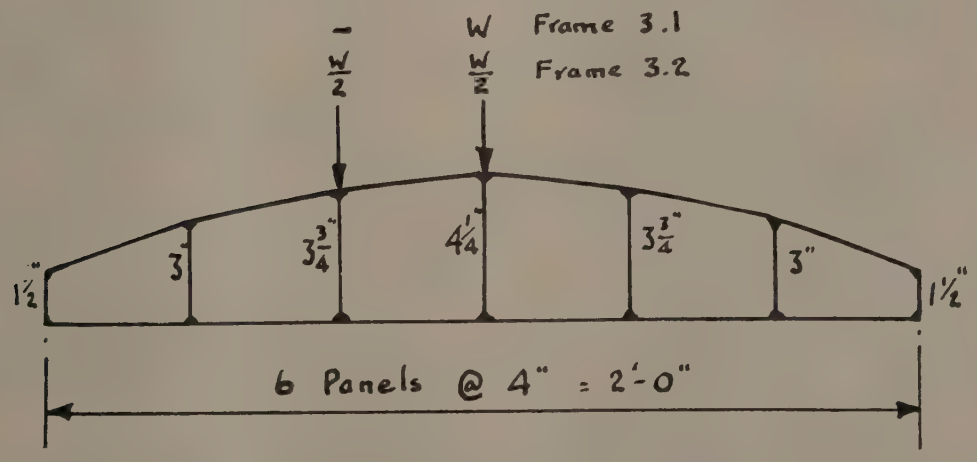


Fig. 10.—Outline diagram of Frame Nos. 3.1 and 3.2. Detail design as for Group I and II frames

might fail by mode A) so that the figure quoted in Table II for failure according to each of the three modes is the minimum value in each case. The appearance of the frames after test confirmed the mode of failure as may be seen in Fig. 2 in which the lowermost frame in the photograph is No. 3.1. The load-deflection curves again indicated that strain hardening had taken place and this would account for the experimental collapse loads exceeding the calculated ones.

The results of these experiments may be considered satisfactory and it may be concluded that a reasonable estimate can be made of the collapse load of a Vierendeel girder.

The Plastic Design of Vierendeel Girders

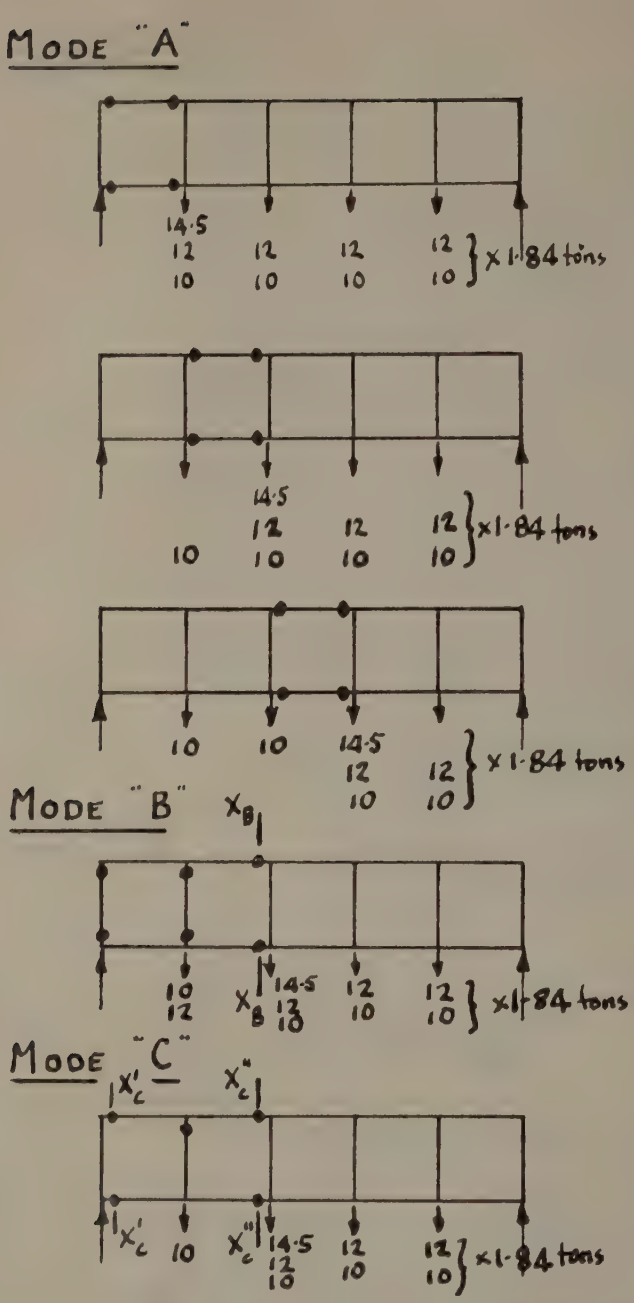
The application of the plastic theory to the design of Vierendeel girders may now be considered. A suitable procedure is to design the chord members against failure by mode A then to design the verticals against failure by modes B and C. This may be illustrated by considering the design of a 50 ft. span girder carrying a dead load of 1 ton/ft. run and a live load of 1.2 tons/ft. together with a knife edge load of 14.5 tons. Assuming that the girder is to have five 10 ft. panels and a maximum depth of 10 ft. the panel shears and moments for various load positions are shown in Fig. 11 for a load factor of 1.84. This part of the work is the same irrespective of the shape of the girder. We may now examine the design of a parallel chord and an inclined chord truss for this loading assuming that uniform chord sections are adopted throughout the length of each girder.

(*a*) *Parallel chord truss.* If as proposed, the chord section is first to be designed against failure by mode A it is obvious that the design of the end panels, which carry the heaviest shear, will determine the chord section.

From Fig. 11 and equation (1) :

$$\frac{p}{2} \cdot S_x = 2 m_p \qquad \therefore m_p = \frac{5 \times 102.4}{2.0}$$

$$= 256 \text{ ft. tons.}$$



Max. Shear = 102.4 tons
$_AM_x$ = 512 ft. tons

Max. Shear = 56.3 tons
$_AM_x$ = 1,208 ft. tons

Max. Shear = 10.7 tons
$_AM_x$ = 1,496 ft. tons

$_BM_x$ = 1,508 ft. tons

$_CM'_x$ = 59.3 ft. tons
$_CM''_x$ = 1,422·5 ft. tons

Fig. 11. Modes of failure, load positions, shears and moments for design of girder

Taking

$f_y = 15.25$ tons/sq. in. $\quad \mu_p = 256 \times \frac{12}{15.25}$

$= 202$ ins.^{3}

Section : 2 Flange plates 12″ × ¾″	170.0	18.0
Web plate 18″ × ½″	40.5	9.0
	μ = 210.5ins.^{3}	A = 27.0ins.^{2}

Axial load in member $= \frac{512}{10} = 51.2$ tons.

Axial stress $= \frac{51.2}{27} = 1.9$ tons/sq. in.

For this axial stress the effective plastic modulus is

$\mu' = 210.5 \times 0.96 = 202.4$ ins.^{3}

Therefore this section is suitable.

It is evident by inspection that the section of th first (and last) vertical must not be less than that c the chord members in the last panel if these are t develop their full plastic moments at the top an bottom of the first vertical. The worst loading con dition for the second vertical for failure by mode occurs when the span is fully loaded and the 14.5 to load is at the third vertical. The support reactio under the factored loads is then 97 tons and the axi

load in the end vertical 48.5 tons producing an axial stress of $\frac{48.5}{27} = 1.8$ tons/sq. in. Then $k = 0.96$ and $\mu' = 210.5 \times 0.96 = 206$ ins^3. $M_p = \frac{206 \times 15.25}{12}$ =262 ft. tons. Allowing that the critical section in the vertical will be 9.75″ from the centre line of the chord member, the maximum horizontal force which can be developed at mid height of this member is therefore :

$$H = \frac{262}{5 - 0.81} = 62.6 \text{ tons.}$$

Applying equation (2) :

$$M_x = F \cdot h + 2 \cdot {}_cM_p'$$

The value of F must be known before the reduced plastic moment ${}_cM_p'$ can be calculated but as a first approximation calculate this taking the axial load on the chords as $\frac{M_x}{h}$.

$$M_x = 1508 \text{ ft. tons.}$$

$$f = \frac{1508}{10 \times 27} = 5.58 \text{ tons/sq. in.} \quad k = 0.75$$

and

$${}_cM_p' = 0.75 \times 210.5 \times \frac{15.25}{12} = 200 \text{ ft. tons.}$$

For the first vertical $H = 62.6$ tons

$$\therefore 1508 = \left(62.6 + \frac{{}_2M_p}{4.19}\right) 10 + 2 \times 200$$

From which :

$${}_2M_p' = 205 \text{ ft. tons.}$$

For failure of the first two panels by mode C we have from equation (3) :

$$\frac{1422.5 - 2 \times 200}{10} - \frac{59.3 + 2 \times 200}{10} = \frac{2 \cdot {}_2M_p'}{8.5}$$

giving ${}_2M_p' = 239$ ft. tons. The net plastic modulus required for the second vertical is therefore $\frac{239 \times 12}{15.25}$ = 188 ins^3.

Try :

2 Flange plates	12″ × ¾″	160
Web plate	17″ × ½″	36
		$\mu = 196$ ins.^{3}

The axial stress in the vertical is small and therefore this section would be suitable. A similar calculation can be carried out for the third vertical.

(*b*) *Inclined chord truss.* The design of a girder with an inclined upper chord in which both chords have the same section throughout proceeds in the following manner.

The chord members are again proportioned against failure by mode A using equation (1.a). If the chord section is kept constant along its length several trials will be necessary to find suitable values for the heights of the verticals—if these are fixed arbitrarily in advance it will in general be found that a uniform load factor against failure by mode A will not be obtained for the several panels of the truss. Thus by trial and error we obtain :

1st Panel

$h_1 = 2.0$ ft. $\quad h_2 = 8.0$ ft.

$$5\left(102.4 - \frac{510 \times 2}{10} \cdot \frac{8-2}{10}\right) = 2 \cdot {}_cM_p'$$

$${}_cM_p' = 41.4 \times 2.5 = 103.5 \text{ ft. tons.}$$

$$\mu = 103.5 \times \frac{12}{15.25} = 81.5 \text{ ins.}^3$$

Section :

2 Flange plates	10″ × ¾″	95.8	15.0
Web plate	13″ × ½″	18.0	6.0
		$\mu = 113.8$ ins.^{3}	$A = 21.0$ ins.^{2}

$$F_1' = \frac{512}{5} \times 0.86 + 51.2 \times 0.51 = 87.6 + 26.1$$

$$= 113.7 \text{ tons.}$$

$$f_1 = \frac{113.7}{21.0} = 5.4 \text{ tons/sq. in.}$$

$k = 0.77 \quad \mu' = 87.5$

$\therefore$ This section is suitable.

2nd Panel

$h_2 = 8.0$ ft. $\quad h_3 = 10.0$ ft.

$$5\left(61.1 - \frac{915 \times 2}{10} \cdot \frac{10-8}{18}\right) = 2 \cdot {}_cM_p'$$

$${}_cM_p' = 40.7 \times 2.5 = 102.0 \text{ ft. tons.}$$

$$F_2' = \frac{915 \times 2}{9.0} \times 0.98 + 30.50 \times 0.2$$

$$= 99.6 + 6.1$$

$$= 105.5 \text{ tons.}$$

$\therefore$ Same section is suitable.

3rd Panel

$$5 \times 23.95 = 2 \cdot {}_cM_p'$$

$${}_cM_p' = 2.5 \times 23.95 = 59.8 \text{ ft. tons.}$$

$$F_3 = \frac{115.0}{10} = 115.0 \text{ tons.}$$

$\therefore$ Same section is suitable.

1st Vertical. The maximum horizontal load on the first vertical occurs when the span is fully loaded and with the 14.5 ton load at the second vertical, i.e. the same position as for the design of the first panel chord members. The horizontal load is then $\frac{512}{5} = 102$ tons. If the same section is used as for the chord members it will be capable of resisting this horizontal load together with half of the end reaction amounting to 51.2 tons.

2nd Vertical. Considering first failure according to mode B the most unfavourable load position is with the span fully loaded and the knife edge load at the third

vertical. The end reaction is then 97.0 tons and the axial stress in the end vertical $\frac{48.5}{21.0} = 2.3$ tons/sq. in. The moment of resistance of the end vertical section is then $0.94 \times 113.8 \times \frac{15.25}{12} = 136$ ft. tons and the maximum horizontal force at mid-height is $\frac{136}{1.0} = 136$ tons. Applying equation (2) :

$$1508 = \left(136 + \frac{{}_2M_p'}{4}\right) . \ 10 + 2 \times 91$$

since ${}_BM_x = 1508$ ft. tons.

and ${}_cM_p' = 0.63 \times 113.8 \times \frac{15.25}{12} = 91$ ft. tons

the axial stress in the chord being $\frac{1508}{10 \times 21.0} = 7.2$ tons/sq. in. This gives a negative result for ${}_2M_p$ indicating that the end vertical is in itself capable of providing the necessary horizontal force.

If failure of panels 1 and 2 by mode C is considered we have :

$$\frac{1422.5 - 2 \times 0.7 \times 113.8}{10}$$

$$- \frac{59.3 + 2 \times 0.86 \times 113.8}{2} = \frac{2 \ . \ {}_2M_p'}{8}$$

This again gives a negative value for ${}_2M_p'$ indicating that failure will not take place by this mode in these two panels under the loading considered. It would appear that the minimum practical section taking into account appearance, connection of cross girders etc. will be sufficient for this member. A similar calculation can be carried out for the design of the third vertical.

In both the above designs it is assumed that buckling of the upper chord members will not take place. This appears to be justifiable as the axial stress in the members at collapse in no case exceeded 8.0 tons/sq. in. with a lateral l/k between panel points of about 34.

The above example is of course a very simple o With a larger number of panels the procedure wo be the same, i.e. the chords would first be designed considering failure of each panel by mode A. T verticals are then designed for failure by mode B a checked for failure by mode C. The number of possi patterns of failure by mode C increases rapidly with t number of panels but most of these can be eliminat by inspection without actual substitution in equati (3).

Conclusion

As compared with the corresponding elastic analy the plastic method is very simple and rapid and adoption would permit the examination of alternati designs without undue expenditure of time. A furth advantage of the plastic method lies in the fact th accurate allowance can be made for the effect of t connections on the strength of the structure ; this po has been fully examined for portal frame structures a previous paper by the author[4] and the general pr ciples are the same for Vierendeel girders. It may observed however that, although the method outlin in this paper possesses these advantages, there can no question of blind substitution in formulae and it essential that the designer should be thorough familiar with the principles and limitations of t plastic method.

References

1. Baes, L., " Analysis of the Vierendeel Girder," *L'Ossat Metallique*, October 1936 and March 1937.
2. Hendry, A. W., " The Simple Plastic Theory," *Civil Engina ing*, March—June, 1950.
3. Hendry, A. W., " An Investigation of the Strength of Cert Welded Portal Frames in Relation to the Plastic Method Design," *Structural Engineer*, Vol. XXVIII, No. 12, 1950.
4. Hendry, A. W., " An Investigation of the Strength of Welc Portal Frame Connections," *Structural Engineer*, Vol. XXV No. 10, October, 1950.

Appendix

Reduction of Plastic Moment by Axial Stress for Yield Point of 15.25 tons/sq. in.

f	0.0	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9
k	1.0	0.98	0.95	0.91	0.86	0.80	0.74	0.67	0.60	0
f	10.0	11.0	12.0	13.0	14.0	15.25				
k	0.43	0.35	0.27	0.18	0.10	0.0				

Book Reviews

(*continued from page 212*)

Strength of Materials (2nd Edition), by Joseph Marin and John A. Sauer. (Macmillan Co., New York, 1954.) 518 pp., 9½in. × 6 in. Price 35s.

This volume comprises a second edition of a 1948 publication, and the authors set out to provide for students of engineering a clear account of the science of mechanics of materials and its application to the simpler problems of design.

The book is divided into four main sections, dealing with

(1) Simple stresses ;
(2) Statically indeterminate stresses ;
(3) Combined stresses ;
(4) Special topics ;

and as would be expected the first three sections cov the same ground as many other equally admirat volumes by earlier authors. The whole of this porti is well and clearly illustrated, and the subject-matt is adequately dealt with.

The section on special topics commends itself to t writer as containing a good deal that is new in the stu of the subject, and the portions dealing with impa fatigue, plastic bending beyond the yield point, cree and experimental test analysis are worthy of spec mention.

Students preparing for the Institution's examinatio will find this volume a useful addition to their referen books, and will appreciate to the full the clarity illustration which is a feature of this publication.

D. T. W.

A Note on the Problem of Rapid Design of Multi-Storey Frames*

By R. H. Wood, Ph.D., B.Sc., A.M.I.Struct.E.
(of the Building Research Station)

General

RAPID design of multi-storey frames has received attention mainly as a result of new techniques, such as limit analysis, electronic and analogue computers, developments in nomography and so on. The elaborate results of present-day research apparently conflict with the stringent demands for rapid design, and it may be helpful in view of correspondence[1] on a previous paper by the author[2] to consider the basic factors that influence the formulation of rapid design methods.

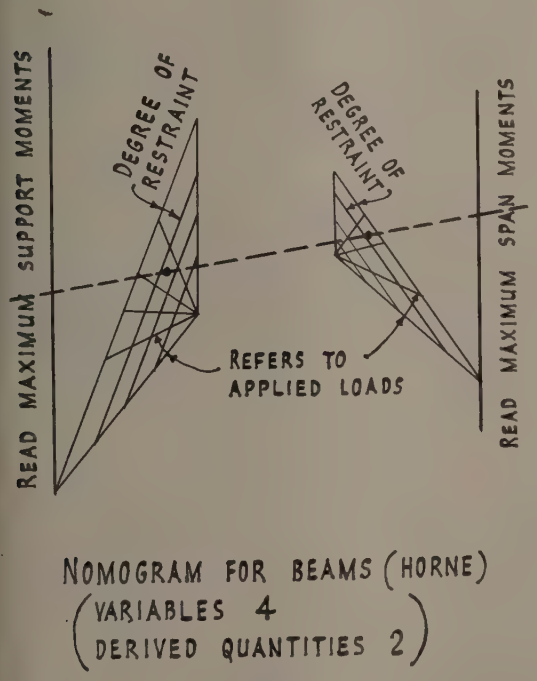


Fig. 1

Rapid design methods must at least satisfy the standards of speed demanded of the Steel Structures Research Committee, whose Draft Rules[3] consist mainly of charts and tables. Relaxation methods of analysis were known to the Committee, but the Draft Rules set an altogether higher standard of speed. Liberal use had been made of idealised substitute frames for which *the results could be known in advance.* By using more elaborate substitute frames the author[4] was able to effect greater economy without loss of speed. This approach is nevertheless limited by the number of variables which can be handled by any one chart or table, and provided sufficiently reliable substitute frames can be devised the Nomogram (Fig. 1) is probably without rival in this respect (vide Horne[5]). The author has now extended this procedure to include instability effects.[2] Since the Nomograms give almost instantaneous solutions, use of a relaxation process is a waste of time providing the substitute frame is sufficiently representative.

The Instability Problem

(*a*) *No Sway.* Even when the sway is prevented by the cladding (e.g. as assumed in the Draft Rules), the problem of formulating a rapid design method involves a dilemma, namely that it is impossible to make separate use of tables of stanchion moments and stanchion design charts, even if the latter charts include the buckling parameter

$$\sqrt{\frac{P}{EI}} \cdot L,$$

without sacrificing the increased buckling loads associated with the concept of the " effective length."

For an initially straight stanchion (Fig. 3)

$$y = -\frac{M_A}{P}\left[\frac{L-x}{L} - \frac{Sin\ \alpha(L-x)}{Sin\ \alpha L}\right] - \frac{M_B}{P}\left[\frac{Sin\ \alpha x}{Sin\ \alpha L} - \frac{x}{L}\right]$$

and at the Euler Load, or $\alpha L = \pi$, the deflections apparently always become infinite. But if we put $\alpha L = \pi - \delta$, where δ tends towards zero, it can be shown that the only possible stable condition is when $M_A = M_B$, i.e. a state of Double Curvature with equal end moments (but not necessarily equal rotations). Therefore tables of varying pre-calculated stanchion moments, taken together with *separate* design charts (refs.[3], [4]), cannot possibly permit higher buckling loads than that of a pin-ended strut. These separate processes must be combined somehow, and the writer knows of no other method of doing this, *without losing speed*, except that of the nomogram, Fig. 2. It cannot be claimed that this method is more than a very powerful computational aid, but it leaves some scope for the designer's own experience.

(*b*) *Sway allowed.* The problem of overall instability of the complete frame[6] can be acute when sway is not prevented. The load factor at which the whole frame becomes elastically unstable when sway occurs is frequently of the order of one-fifth of that for individual

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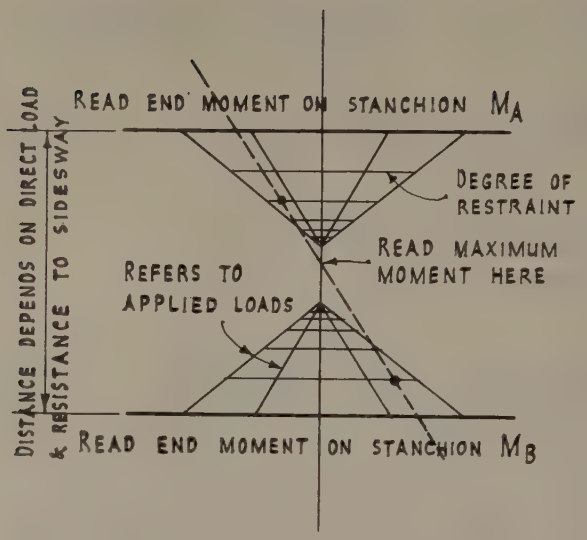

READ END MOMENT ON STANCHION M_A
DEGREE OF RESTRAINT
READ MAXIMUM MOMENT HERE
REFERS TO APPLIED LOADS
DISTANCE DEPENDS ON DIRECT LOAD & RESISTANCE TO SIDESWAY
READ END MOMENT ON STANCHION M_B
NOMOGRAM FOR STANCHIONS (WOOD)
(VARIABLES 4
DERIVED QUANTITIES 3)

Fig. 2

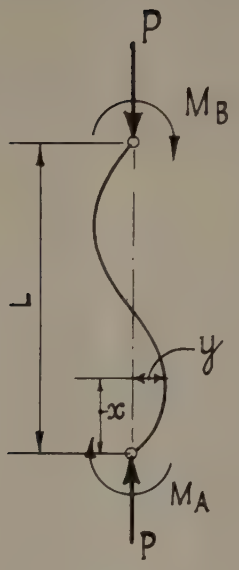


Fig. 3

stanchions when sway is prevented, and plasticity can reduce the instability load still further[7]. There is no satisfactory rapid general method as yet.

Summary

Excepting Elasto-Plastic theories which are still in the process of development, the following table gives a list of existing rapid methods, of sufficient generality and simplicity. They are arranged in order of economy.

References

[1] Correspondence, by W. Merchant and A. Bolton, *J.I.Struct.E.*, August, 1954, p. 230.
[2] R. H. Wood. "A Derivation of Maximum Stanchion Moments in Multi-Storey Frames by means of Nomograms," *J.I.Struct.E.*, November, 1953.
[3] Final Report, *Steel Structures Research Committee*, H.M.S.O., 1936.
[4] R. H. Wood, "An Economical Design of Rigid Steel Frames for Multi-Storey Buildings," *National Building Studies Research Paper No.* 10, H.M.S.O., 1951.
[5] M. R. Horne, "Maximum Beam Moments in Welded Building Frames," *J.I.Struct.E.*, May, 1950.
[6] W. Merchant, "The Failure Load of Rigid Jointed Frame works as influenced by Stability," *J.I.Struct.E.*, July, 1954.
[7] Contribution by Author to Discussion on "Symposium on High Buildings," *Inst. Civil Engineers*, December, 1954.

Refs.	Author	Beams	Stanchions	Conditions of Validity†	Remarks
3	S.S.R.C.	Draft Rules, Charts	Draft Rules, Tables	No Sway	Semi-Rigid Connections.
3, 5	S.S.R.C. / Horne	Nomograms	Draft Rules, Tables	No Sway	Rigid Frames. No allowance for equivalent length in stanchions.
4	Wood	Design Charts	Design Charts	No Sway	Ditto. Some allowance for moment redistribution.
5. 2	Horne / Wood	Nomograms	Nomograms	No Sway	Very rapid and economical, but requires some knowledge of worst loading arrangements for stanchions. Equivalent length effects included.

† Methods of dealing with wind loads are left to the discretion of the designer.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1., on Wednesday, 25th May, 1955, at 5.55 p.m., Dr. S. B. Hamilton, M.Sc., B.Sc.(Eng.), A.R.C.S., M.I.C.E., M.I.Struct.E. (President) in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

Students

Brockwell, Keith, of Salisbury, Southern Rhodesia.
Fawcett, Frank Birkinshaw, of Ealand, nr. Scunthorpe, Lincs.
Lefever, Geoffrey James, of London.
Parrott, John Frank, of Scunthorpe, Lincs.
Shanmukhanandam, Thiagaraja, of Sale, Cheshire.
Sim Bee Teck, of Singapore.
Stanmore, Anthony Thomas, of London.
Tune, Donald Hector, of Scunthorpe, Lincs.

Graduates

Bacon, Roger, B.Sc.(Eng.) London, of Epsom, Surrey.
Beksinski, Jerzy, of London.
Bent, George Deryk, B.Sc.(Civil) Manchester, of Warrington, Lancs.
Booth, James Gerrard, B.Sc.(Tech.) Manchester, of Bolton, Lancs.
Bradbury, John Norman, B.Sc.(Civil) Birmingham, of London.
Chow, David Siong Keng, B.Sc. California, D.I.C., of London.
Cohen, Maurice, B.Sc.(Eng.) London, of London.
Cornfoot, William Bright, B.Sc.(Eng.) Durham, of Whitley Bay, Northumberland.
Day, Alistair Scott, B.Sc.(Eng.) Bristol, of Bristol.
Dixon, Francis George, A.R.I.B.A., of Welwyn Garden City, Herts.
Eldridge, Dennis Kenneth, of Whitwick, Leics.
Erntroy, Henry Chanoch, M.Sc.(Eng.) London, of London.
Eu Sin Han., B.Sc., D.I.C., of Nottingham.
Flower, Brian Langfield, of Thames Ditton, Surrey.
Fung Yuen-Sin, B.Sc. Hong Kong, D.I.C., of London.
Golebiewski, Alfons, of London.
Griffin, Bryan Edward, of Staines, Middlesex.
Hanna, Albert Michael, of London.
Horne, Michael John Barton, of Pulborough, Sussex.
Kwiecinski, Stanislaw, of London.
Leveridge, Peter James, A.M.I.Mun.E., of Harlow, Essex.
Lowe, Kenneth Herbert, B.Sc.(Civil) Cape Town, of Livingstone, Northern Rhodesia.
McMahon, Bernard Michael, B.E., N.U.I., of Stafford.
Masterman, Ivor John, B.Sc.(Eng.) London, of London.
Mitchell, John Philip, M.Sc. Leeds, of Liverpool.
Murphy, Kevin Malcolm, B.E. Western Australia, of London.
Ostoja-Kotkowski, Maciej Zygmunt Scibor, of London.
Rees, Michael Howell, B.Sc.(Eng.) Rand., of London.
Rynkiewicz, Leonard Stanislaw, B.Sc.(Civil) Natal, of Durban, Natal, South Africa.
Savage, Derek William, of Lanesfield, nr. Wolverhampton, Staffs.
Smith, Kenneth Vivian Hewson, M.Sc.(Eng.) London, of Nicosia, Cyprus.
Stainsby, Richard, of Birmingham.
Stephens, Robert Peter, B.Sc.(Eng.) London, of Exeter, Devon.
Strong, John Wallace, of Middlesbrough, Yorks.
Tomaszewski, Henryk, of London.
Tulloch, Joseph Samuel, of London.
Venter, Jacobus Johannes, B.Sc.(Eng.) Rand., of Livingstone, Northern Rhodesia.
Watts, John Sanderson, of Wirral, Cheshire.
Wright, William Hector Charles, B.Sc. Glasgow, of Woking, Surrey.

Member

Pattison, Thomas, of Johannesburg, South Africa.

Transfers

Students to Graduates

Allaway, Godfrey Walter, of Stockton-on-Tees, Co. Durham.
Carter, Clifford, of Salford, Lancs.
Dudzinski, Tadeusz, of London.
Kemp, Leslie William, of London.
McCarthy, Patrick O'Gorman, of London.
McGladdery, Alan Wilson, of Cowley, Middlesex.
Middleton, James Henry, of Manchester.
Rimmer, William, of Burtonwood, nr. Warrington, Lancs.
Thompson, Gordon, of Manchester .
Whitaker, Norman, of Ramsbottom, nr. Manchester.

Graduates to Associate-Members

Bettany, George Angus, of South Benfleet, Essex.
Booth, Alan Sutherland, of Salford, Lancs.
Bullock, Francis Hubert, B.A. Cantab., of London.
Dale, John Granton, of Liverpool.
Davies, David Charles, B.Sc.(Eng.) London, of Whitstable, Kent.
Denson, Francis Ronald, of Bolton, Lancs.
Hardy, Donald, of Elland, Yorks.
Horton, William John, B.Sc.(Eng.) Cape Town, of Langstone, nr. Havant, Hampshire.
Lord, Peter Bamford, of Derby.
Richards, Gerald William, of Gnosall, Stafford.
Robinson, William Whitfield, of Liverpool.
Vaswani, Harkrishin Pahlajrai, B.E. Bombay, of Goteborg, Sweden.
Walmsley, Joseph Roy, of Bolton, Lancs.

Associate-Members to Members

FORSYTH, John Macdonald Gordon, B.Sc.(Eng.) London, A.M.I.C.E., of Horsham, Sussex.

MCLEAN, Ronald Alfred Milner, B.Sc.(Eng.) Belfast, A.M.I.C.E., of Bulawayo, Southern Rhodesia.

ROBINSON, James Parkin, A.M.I.C.E., of Harrogate, Yorks.

WUEST, Henry George, of Orpington, Kent.

RE-ADMISSION

Graduate

ARCEIVALA, Solie Jal, of Cambridge, Massachusetts, U.S.A.

WITHDRAWAL OF RESIGNATION

The Council have accepted the withdrawal of the resignation of Major P. B. BRYDEN, of Wellington, New Zealand.

OBITUARY

The Council regret to announce the deaths of Sir Edwin AIREY, George Wesley COSTAIN, John Frederick FARQUHARSON, Richard Bertram LING, Thomas James MCDONALD (Members); Frank Quentery FARMER, William John WILLIAMS (Retired Members); Lawrence LEE-DUNHAM (Associate-Member).

ANNUAL GENERAL MEETING

The Annual General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1., on Wednesday, May 25th, 1955, at 6 p.m., Dr. S. B. Hamilton, M.Sc., B.Sc.(Eng.), A.R.C.S., M.I.C.E., M.I.Struct.E., in the Chair.

The Secretary (Major R. F. Maitland, O.B.E.) read the notice convening the meeting.

The Minutes of the Annual General Meeting held on the 27th May, 1954, as published in THE STRUCTURAL ENGINEER, July, 1954, were taken as read and were confirmed and signed.

Mr. D. T. Williams (Associate-Member of Council) moved the adoption of the Sessional Report of the Council and the accounts for the financial year, 1954. Mr. E. N. Underwood (Delegate Member of Council) seconded the motion, which was carried unanimously.

Lt.-Col. G. W. Kirkland, M.B.E. (Hon. Curator) proposed the re-election of Messrs. James Meston & Co. Chartered Accountants, as Auditors for the ensuing year. Mr. W. H. Woodcock (Vice-President) seconded the motion, which was carried unanimously.

The Secretary then read the report of the Scrutineers on the ballot for the election of President, the Honorary Officers and the Ordinary members of Council for the Session 1955-6, as follows :—

To : The Council of the Institution of Structural Engineers.

Gentlemen :

We, the undersigned, report that at the request of the President we have duly carried out the duties of Scrutineers of the Ballot for the election of Honorary Officers and Council for the Session 1955-56, and we report accordingly as follows :

We received 802 Ballot Papers, of which we rejected 90 as wholly spoiled and 10 as partly spoiled. We have attached a separate sheet showing the number of votes received by each candidate.

We declare the result of the Ballot to be as follows :

Elected

PRESIDENT—Mr. Stanley Vaughan, B.Sc., M.I.C.E., A.C.G.I.

VICE-PRESIDENTS—Mr. J. Guthrie Brown, M.I.C.E.; Professor A. G. Pugsley, O.B.E., D.Sc., F.R.S., M.I.C.E., A.M.Am.Soc.C.E., F.R.Ae.S.; Mr. G. S. McDonald, M.I.C.E.; Mr. L. E. Kent, B.Sc.(Eng.), M.I.C.E.; Mr. W. H. Woodcock, F.C.S.; Professor A. J. Ockleston, B.E., Ph.D., D.Sc., M.I.C.E.

HONORARY TREASURER—Mr. E. N. Underwood, B.Sc.(Eng.), M.I.C.E.

HONORARY SECRETARY—Mr. F. R. Bullen, B.Sc.(Eng.), M.I.C.E.

HONORARY LIBRARIAN—Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E.

HONORARY EDITOR—Lt.-Col. G. W. Kirkland, M.B.E.

HONORARY CURATOR—Mr. R. Summers.

The above are all elected for ONE YEAR.

ELECTED AS ORDINARY MEMBERS OF COUNCIL (LONDON) —Mr. T. F. Burns, M.I.C.E.; Mr. F. M. Bowen, M.I.C.E., Assoc.I.Mech.E.; Mr. R. W. Schofield.

The above are elected for THREE YEARS.

ELECTED AS ORDINARY MEMBER OF COUNCIL (COUNTRY)—Sir Donald Bailey, O.B.E., J.P., D.Eng., A.M.I.C.E., Hon. M. Institution of Royal Engineers.

The above is elected for THREE YEARS.

ELECTED AS ASSOCIATE-MEMBER OF COUNCIL (LONDON) —Mr. R. C. Buxton, B.A.(Cantab).

The above is elected for THREE YEARS.

ELECTED AS ASSOCIATE-MEMBER OF COUNCIL (COUNTRY)—Mr. A. S. Sinclair.

The above is elected for THREE YEARS.

We are, Gentlemen,

Yours faithfully (signed) A. Raymond Mail, J. E. L. De Bremaeker, Gilbert Roberts, Douglas V. Pike (Scrutineers).

On a motion by the President, a vote of thanks was unanimously passed to the scrutineers.

LANCASHIRE AND CHESHIRE BRANCH

Mr. J. L. Robinson has succeeded Mr. A. S. Sinclair as Joint Honorary Secretary of the Lancashire and Cheshire Branch. Mr. Sinclair held office as Honorary Secretary of the Branch since 1951 and in this capacity has rendered valuable service to the Institution.

EXAMINATIONS, JULY, 1955

The Examinations of the Institution will next be held at centres in the United Kingdom and Overseas on the 12th and 13th July, 1955 (Graduate) and the 14th and 15th July (Associate-Membership).

REPRESENTATION OVERSEAS

The Council have appointed Mr. E. F. Cornillie (Member) to act as the Institution's Representative in the British West Indies in place of Mr. C. Dupenois, who has left the Colony.

ENGINEERING MARINE AND WELDING EXHIBITION

The Directors of the Engineering, Marine and Welding Exhibition, which will be held at Olympia from the 1st to 15th September, 1955, have invited all members of the Institution to visit the Exhibition on Tuesday, 13th September, 1955. A ticket of admission, available for two persons, which may be used for the official visit or on any other single day, may be obtained on application to the Secretary.

DRURY MEDAL AWARD

The fifth competition for the above award will take place in 1955. The subject is the design of a mobile crane.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary : envelopes to be marked in the top left-hand corner, " Drury Medal Award."

The closing date for the competition is October 1st, 1955.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say, primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution, or failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate in the presence of a Justice of the Peace or a Commissioner for Oaths.

RESEARCH AWARDS

The Council have instituted a Research Prize Fund, from which awards may be made annually to the author or joint authors of papers describing original research which they have carried out. Research awards may be made for papers read at Headquarters or in the Branches and published in the Journal, or for papers published in the Journal only without being read at an open meeting.

The assessment for such awards will be made annually, but awards will be made only to the contributors of such papers as reach a standard judged by the Literature Committee to be satisfactory.

Work submitted under this scheme must be original and may include any of the following :

(a) investigations of an experimental or analytical character ;
(b) studies of historical or statistical records ;
(c) improvements in principles or methods of construction ;
(d) research into methods of structural engineering and building, the nature and use of plant and the organisation of engineering work ;
(e) any related or combined studies which are deemed by the Literature Committee to be of a research character.

In cases where the research work described in the paper was not the work of one individual, the names of all the collaborators should be given in the paper.

Awards may take any or all of the following forms : A research medal ; a diploma ; a money prize.

Application for consideration for a research award must be made to the Secretary of the Institution, and in preparing papers for reproduction in the Journal, authors must comply with the conditions laid down for all such contributions. Particulars of these conditions may be obtained from the Secretary.

In judging research papers, the following factors will be considered :

(a) the nature of the subject and its conclusions ;
(b) the value of the paper in advancing the science and art of structural engineering ;
(c) the standard of preparation and orderly arrangement of the subject-matter.

Research papers will also be eligible for adjudication for the Institution Medals if they comply with the Regulations governing those awards.

The closing date for the receipt of applications in respect of papers published in the Journal between October, 1954, and September, 1955, is October 3rd, 1955.

YEAR BOOK AND LIST OF MEMBERS

The Year Book and List of Members for 1955 will go to press in July, for publication in October, when a copy will be sent to all members.

Members are requested to inform the Secretary of any alterations in titles, degrees or addresses, which have not already been notified, by July 4th, in order that such amendments may be included in the new edition.

REPRINTS FROM " THE STRUCTURAL ENGINEER "

Authors who wish to obtain reprints of their papers published in " The Structural Engineer " should notify the Secretary of the Institution not later than the 15th of the month in which the paper is published.

LONDON GRADUATES' AND STUDENTS' SECTION

At the Extraordinary Annual General Meeting held at 11, Upper Belgrave Street, London, S.W.1., on Monday, 9th May, 1955, the following Honorary Officers and Committee were elected for the Session 1955-6 :

Chairman : Mr. J. A. Pope.

Vice-Chairman : Mr. B. J. White.

Hon. Secretary : Mr. D. E. Capelin, 60, Thornton Crescent, Old Coulsdon, Surrey.

Hon. Treasurer : Mr. J. F. S. Pryke.

Committee : Messrs. S. B. Tietz, J. Churcher, P. S. Coleman.

A visit to the works of the Tunnel Cement Company Ltd., at West Thurrock has been arranged for Saturday, 23rd July. The coach will leave 11, Upper Belgrave Street at 8.45 a.m., and as the party is limited it is requested that all who wish to take part in the visit should communicate with Mr. Pope.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The Annual General Business Meeting of the Branch was held in Manchester on the 28th April, when the following Honorary Officers and Committee members were elected for the Session 1955-6 :

Chairman : Mr. J. H. Morris.
Vice-Chairman : Mr. J. B. G. Martin.

Immediate Past Chairman : Mr. W. D. Blades.
Honorary Auditors : Mr. K. Norrey, Mr. G. S. Jones.
Joint Honorary Secretaries : Mr. J. L. Robinson, 314, Northenden Road, Sale, Manchester ; Mr. M. D. Woods, 58, Spring Gardens, Salford, 6.
Hon. Assistant Secretary : Mr. J. Parr.
Committee : Dr. D. D. Matthews, Messrs. W. Fitton, S. Gleaves, R. Gray (Past Chairman), W. Bates (Past Chairman), F. C. Brookhouse (Past Chairman), Professor J. A. L. Matheson (Past Chairman), Messrs. G. Greenlees, A. E. Wright, K. Norrey, H. J. Dowling, A. S. Sinclair.

The Annual Report of the Branch for the Session 1954-5 was presented. Thirteen meetings were held during the Session—four in Liverpool, seven in Manchester, and one in Blackpool. Six of the meetings were held jointly with other Societies. In addition, a visit to a site was arranged and a very successful Dinner and Dance was held.

The Branch suffered a severe loss on the death of one of its founders, Mr. H. H. Clapham, who had served on the Committee for many years between 1922 and 1950 and had been Branch Chairman in 1933-4.

MIDLAND COUNTIES BRANCH

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.
Midland Counties Graduates' and Students' Section—
Hon. Secretary : A. K. A. Costain, A.M.I.C.E., 134, Witherford Way, Weoley Hill, Birmingham, 29.

NORTHERN COUNTIES BRANCH

The Annual General Meeting of the Branch was held at Middlesbrough on the 5th April, when the following Honorary Officers and Committee members were elected for the Session 1955-6 :

Chairman : Mr. E. A. Parsons.
Vice-Chairman : Mr. T. Johnson.
Immediate Past Chairman : Mr. W. G. Gentry.
Branch Hon. Secretary : Captain O. Lithgow, 4, Stoneleigh Avenue, Acklam, Middlesbrough.
Tyne Centre Hon. Secretary : Mr. J. Whitten, 14, Athol Gardens, Whitley Bay, Northumberland.
Branch Hon. Treasurer : Mr. L. Dobson, Messrs. L. G. Mouchel & Partners, 9, Ellison Place, Newcastle, 1.
Committee : Tees Centre : Messrs. A. Burton, E. Ibbotson, H. Rees, T. R. Tighe, D. J. Leggott ; Tyne Centre : Messrs. D. M. O'Herlihy, E. Atkinson, T. Usherwood, J. B. Armstrong, J. S. Colverd. Ex-Officio Members (Branch Bye-Law No. 4) : Mr. G. S. Gowland and Mr. W. R. Garrett.

NORTHERN IRELAND BRANCH

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., " Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The Annual General Meeting of the Branch was held in Glasgow on the 26th April, when the following Honorary Officers and members of Committee were elected for the Session 1955-6 :

Chairman : Professor W. T. Marshall.
Vice-Chairman : Mr. J. Cameron.
Immediate Past Chairman : Mr. W. Heigh.
Honorary Treasurer : Mr. W. Shearer Smith.
Hon. Auditors : Mr. R. H. Sharpe, Mr. P. M. Plews.
Hon. Secretary : Mr. W. Basil Scott, 19 Waterloo Street, Glasgow, C.2.
Committee : Messrs. J. A. Brown, G. M. Dingwa R. Drummond, R. McD. Howatt, O.B.E., M.C., T.D J. Laird, S. A. Loveland, A. Mason, A. Maclean, A. McNaught, A. G. F. Russell, M. Stark.

SOUTH WESTERN COUNTIES BRANCH

Joint Hon. Secretaries : E. W. Howells, M.I.Struct.E 10/12, Market Street, Torquay, Devon ; C. J. Woodro " Elstow," Hartley Park Villas, Tavistock Roa Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The Annual General Meeting of the Branch was he at Port Talbot on the 3rd May, when the followi Honorary Officers and Committee members we elected for the Session 1955-6 :

Chairman : Mr. A. V. Hooker.
Senior Vice-Chairman : Mr. G. E. Cooper.
Junior Vice-Chairman : Mr. J. Warnock.
Hon. Secretary : Mr. K. J. Stewart, 15, Glanmor Roa Swansea, Glam.
Hon. Assistant Secretaries : North Wales, Mr. S. Brown ; Cardiff, Mr. W. D. Hollyman.
Hon. Auditors : Mr. G. W. Spooner, Mr. E. O. Jone
Committee : Messrs. F. V. M. Bell (Past Chairma G. R. Brueton, J. L. Bannister, W. Evans, Dr. A. Fordham, Messrs. G. H. Hodgson, W. D. Hollyma Lt.-Col. R. D. Heseltine, Messrs. D. A. Lewis, Manolopoulos, H. V. Morris, E. R. Steward, G. Spooner, A. G. Thompson.

WESTERN COUNTIES BRANCH

Hon. Secretary : E. Hughes, M.I.Struct.E., 23, Sout down Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, T Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

The first, and very successful, Annual Dinner-Dan of the Branch was held at the Dorchester Hotel, Ros bank, Johannesburg, on the 1st April, and was attend by 145 members and guests.

At the end of the dinner, the Branch Chairma Mr. C. A. Pringle, presented the certificate of Honora Associateship of the Institution to Professor W. Sutton, who in his reply thanked the Institution for t honour conferred on him and gave a short address the growth and development of the various engineeri institutions.

Among the guests present were Mrs. Sutton, M Leslie (Chief Engineer of the Rand Water Board a President of the Associated Scientific and Technic Societies) and Mrs. Leslie, Mr. Gillespie (President the South African Institution of Production Engineers and Professor Ockleston (President of the South Africa Prestressed Concrete Group) and Mrs. Ockleston.

Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E A.M.I.Struct.E., P.O. Box No. 3306, Johannesbur South Africa.

During weekdays Mr. Tait can be contacted in t City Engineer's Department, Town Hall, Johannesbur Phone 34-1111, Ext. 257.

Natal Hon. Secretary : E. G. Bennett, A.M.I.Struct.E c/o The Reinforcing Steel Co. Ltd., P.O. Box 49, Mer bank, Durban.

Cape Section Hon. Secretary: R. Stubbs, M.I.Struct.E African Guarantee Building, 8, St. George's Street, Ca Town.

The Critical Load of Portal Frames when Sidesway is Permitted

By Arthur Bolton, M.Sc. Tech. (Graduate)

Introduction

The importance of being able to calculate the critical load of a strut has long been recognized, and a considerable part of any modern design code is devoted to this end. However, progressive structural engineers are dissatisfied with a design process in which it is necessary to consider the stability of a stanchion as though it were isolated from the frame. When this is done no exact allowance can be made for the stiffening effect of adjacent members or of the redistribution of moments which takes place as the axial loads in members are increased. It seems likely that a rational design method can be developed using the critical load of the complete frame as one parameter.[1] This paper demonstrates a method of obtaining the critical load of building frames in which sway can take place and includes approximations which are believed to be accurate and quick enough for use in such a design.

Outline of Available Methods

It is interesting to compare available methods of calculating critical loads with those in ordinary rigid frame analysis. Strain Energy was first used to calculate the moments in rigid frames. However, this required integrations along all members to obtain each redundant and the method was impracticable for frames other than the simplest. More practical was the Slope Deflection method in which instead of integrating effects along members for unknown applied forces, the unknowns were taken as the rotations and sways of the various joints. This system of unknowns is easily visualised and the necessary equations can be quickly set up (if a suitable calculating machine is available they can also be easily and accurately solved). Even better are the numerical methods such as Moment Distribution and Relaxation developed from Slope Deflection. These enable an engineer to solve without mechanical help problems otherwise too difficult to be practical, or alternatively to use any available calculating machine to its limit.

Following this same path in stability problems the first solutions were likewise obtained by solving differential equations, and early methods were thus limited to the very simplest problems. Next modifications of the Slope Deflection Formula were developed to take account of axial loads. Originally these were based on trigonometric functions of $\alpha = \frac{\pi}{2}\sqrt{\frac{P}{Pe}}$ and thus were rather difficult to visualise ; for example the carry-over factor in this notation becomes

$$\frac{2\alpha - \sin 2\alpha}{\sin 2\alpha - 2\alpha \cos 2\alpha}$$

$\left[P \text{ is taken to mean the axial load in the member and } Pe \text{ to mean } \frac{\pi^2 EI}{L^2}\right]$

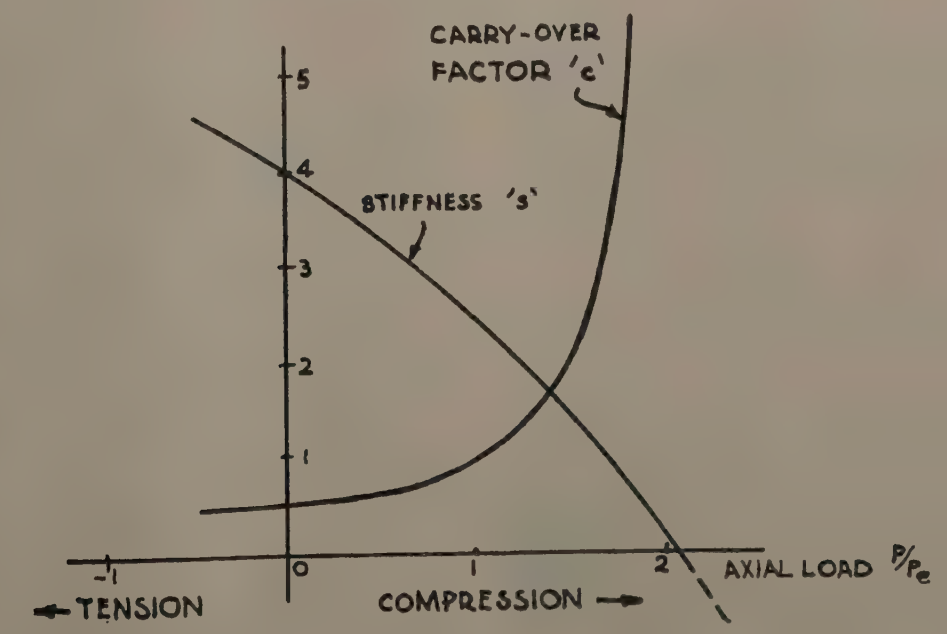


Fig. 1

This meant that the physical behaviour of the structure was often misunderstood. Very easily visualised functions s the effective end stiffness and c the carry over factor were used by James[2] and Lundquist.[3] In this case s gives the effect of axial load on the stiffness of one end of a member against rotation when the other end is fixed, and c the effect on the moment carried over to the fixed end due to this rotation.

The stiffness s is high for large tensile loads, reduces to 4 at zero axial load and to zero at an axial compressive load of just over twice the Euler Load. The carry over factor c increases from a small value for high axial tension to $\frac{1}{2}$ at zero axial load and to infinity at an axial compression of just over twice the Euler Load (Fig. 1). This overall picture is easily remembered and it is possible mentally to forecast the qualitative behaviour of a structure at a given load. Livesley[4] has made the tabulation of s and c far more practical by basing the axial load on the $\frac{P}{Pe}$ value rather than on α and by tabulating other functions of s and c which are very useful in calculation. Without reviewing all the forms of Modified Slope Deflection which have been used it is the writer's opinion that the following gives the clearest physical picture of the distortion

$$M_{AB} = \frac{EI}{L}\left[s\theta_A + sc\theta_B - s\,(1+c)\,\frac{\delta}{L}\right]$$

M_{AB} is the applied moment caused by the given distortions θ_A, θ_B and δ shown in Fig. 2.

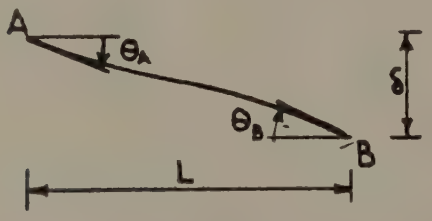


Fig. 2

Using this formula the equations for a structure can be set up and the stiffness against any displacement obtained at any multiple of the design load.

The formula can also be used as the basis of a modified form of Moment Distribution or Relaxation in which the appropriate end stiffnesses and carry-over factors are used.

Use of Tabulated Information—Patterns of moments and forces

1. *Rotation of a joint*

If one joint at which beams and axially loaded members meet is rotated, the moments appearing at the ends of the members are proportional to $k \times s$ where k is the stiffness (I/L). The moment appearing at the far end of members rotated will be proportional to $k \times s \times c$. When the axial load is zero, i.e. for a beam $s = 4$ and $c = \frac{1}{2}$. The horizontal forces associated with these rotations are found as usual by dividing the sum of the moments appearing at the column ends by the storey height. A typical pattern is shown in Fig. 3.

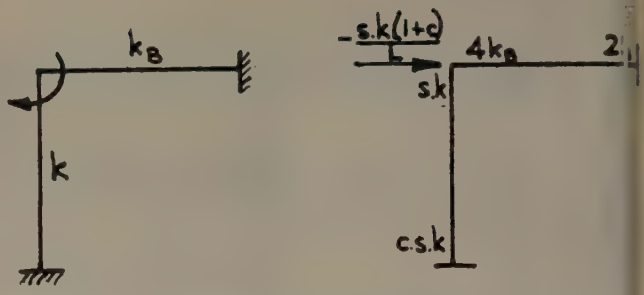


Fig. 3.—Pattern of Moments and Forces due to Rotation of a Joint

2. *Sway of one end of a stanchion relative to the other.*

From the Modified Slope Deflection Formula

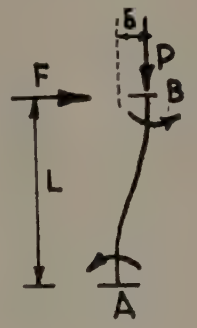


Fig. 4

$$M_{AB} = M_{BA} = \frac{EI}{L}\left[-s\,(1+c)\,\frac{\delta}{L}\right]$$

Shear equation—

$$M_{AB} + M_{BA} + F \times L + P \times \delta = 0$$

$$\therefore F \times L = -(M_{AB} + M_{BA}) -$$

If $M_{AB} = 1$ i.e. $\delta = -\frac{L^2}{EIs\,(1+c)}$

$$F \times L = -2 + \frac{PL^2}{EIs}\,(1+$$

$$= -2 + \frac{P}{Pe} \times \frac{\pi}{s\,(1-$$

$$= -\frac{2}{m}$$

where Merchant's factor[10]

$$m = \frac{1}{1 - \frac{P}{Pe}\,\frac{\pi^2}{2s\,(1+c)}}$$

and is tabulated by Livesley and Chandler.[4]

Hence the pattern of moments and forces caused sway is as shown in Fig. 5.

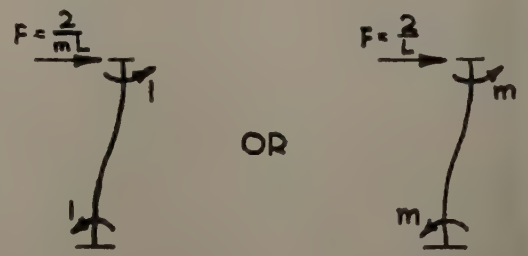


Fig. 5.—Pattern of Moment and Forces due to Swa

If several members sway the same amount the sway oments will be proportional to $\frac{EI\ s(1+c)}{L^2}$, $s(1+c)$ is so tabulated by Livesley.

Fig. 6 shows the derivation of the "No Shear" attern for a symmetrical single bay frame. Fig. 6 (b) hows the pattern of moments and forces due to equal otations of B and C and (c) the pattern for a sway. n 6 (d) these are combined to eliminate the horizontal constraint at B. The expressions

$$s\left[1-\frac{m(1+c)}{2}\right] \quad \text{and} \quad s\left[c-\frac{m(1+c)}{2}\right] \quad \text{have}$$

been tabulated by Livesley as n and $-o$ respectively, so in practice the resulting "No Shear" patterns of Fig. 6 (e) can be written down directly from tables. "No shear" patterns have been found to be very useful in critical load determination and are discussed later.

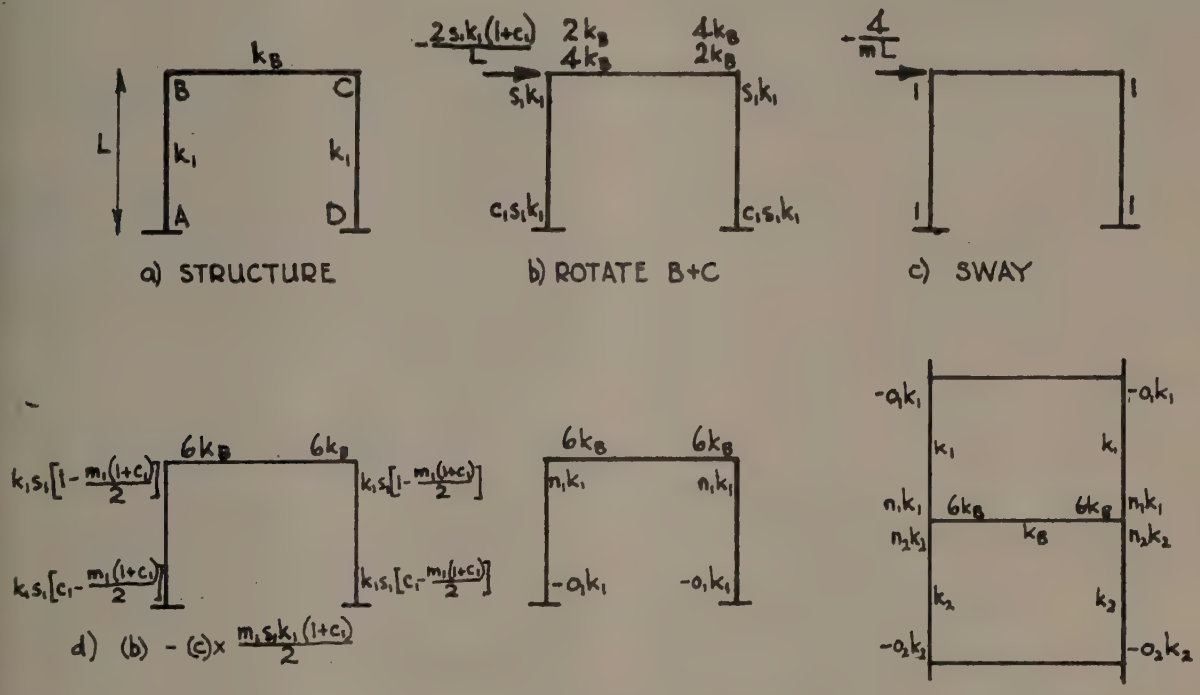


Fig. 6

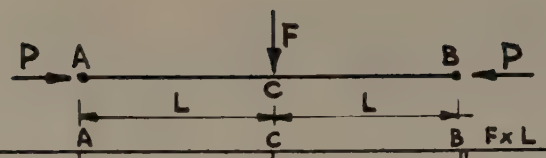
Critical Modes of Failure

Knowing the behaviour of an isolated member under axial load it is possible to find the critical load of a frame in which several members are rigidly connected. This is simplified by the behaviour of a structure at its critical load. The critical load of a structure may be defined as the load at which the structure will not resist some possible distorting force. For instance at the Euler load a pin ended strut will not resist rotation of either end.

ROTATE A	s	sc
BALANCE B	$-sc^2$	$-sc$
TOTAL MOMENTS	$s(1-c^2)$	0

Table 1

But at the Euler load $c = 1$ therefore $s(1-c^2) = 0$ and zero moment M_{AB} is required to rotate A. Neither will the strut resist a force tending to cause lateral displacement of its centre.



	A	C		B	F x L
SWAY C	-m	-m	+m	+m	+4
BALANCE A AND B	+m	+mc	-mc	-m	-2m(1+c)
TOTAL	0	-m(1-c)	+m(1-c)	0	4-2m(1+c)

Table 2

m, L, and c in this calculation relate to half the total length AB so that at the Euler load for AB, m and c will have the values relating to $\frac{P}{Pe} = \frac{1}{4}$ i.e. $m = 1.27$ $c = 0.57$.

Hence at the Euler load for AB

$$F \times L = 4 - 2 \times 1.27\ (1+0.57) = 0$$

Therefore zero transverse force F is required to displace the centre C.

If any disturbing force can be found which a structure will not resist then it is unstable. A point which has caused considerable misunderstanding is that the structure if propped against this particular force may be able to resist some other distortion. This has led in the past to the choice of a distortion which implicitly assumes such propping and then an incorrect decision that the structure is stable.

For instance if equal clockwise moments are applied at both ends of an Euler strut so that it is bent into double curvature, it becomes unstable at the Euler load just as if the end moments were not applied. Yet a simple calculation shows that the strut will apparently resist the combined effect of the two end moments

ROTATE A	s	sc
ROTATE B	sc	s
TOTAL MOMENTS	s(1+c)	s(1+c)

Table 3

At the Euler load $s = 2.47$ $\quad c = 1$

hence $M_{AB} = M_{BA} = 4.94$ and the strut can apparently exert moments to resist the applied end couples.

The reason for this inconsistency is that the real strut deflects laterally at its centre as shown above in Table 2, but in calculating the resistance to end rotations in Table 3 this possibility was ignored, i.e. C was assumed not to deflect. However it is not possible to alter the lowest critical mode merely by considering an alternative deformation in the calculations, the lowest critical mode must be prevented by a definite constraint —in the present case some constraint against the lateral displacement of C. If this is done then the strut will resist the two end couples, or any other small disturbing force.

These simple examples draw attention to the fact that a structure may fail in more than one mode; there are of course an infinite number of these modes. In general only the lowest will be significant since it is impossible to pass the lowest critical load without additional constraints. If the distortion used to test the structure for instability contains any component of the lowest critical mode then the lowest critical value will be the one predicted. If the mode used to test the structure contains no component of the lowest mode as was the case with two equal end moments on the pin ended strut, then a higher and therefore fictitious value will be predicted.

It is important that the distortion used as a test of stability should approximate as closely as possible to the lowest critical mode. If it does so the convergence of the subsequent calculation will be found to be better than when the chosen mode differs widely.

The lowest mode for a building frame built of horizontal beams and vertical stanchions without later bracing, is invariably a sway case since this includes both rotations and sways. The mode obtained by joint rotations without sway merely needs additional horizontal constraints at each floor level. Therefore the testing distortion should be provided by forces tending to sway either one storey or the whole frame.

It is important to consider further the convergence of this type of calculation. In ordinary rigid frame calculations it becomes increasingly laborious to obtain a solution as the size of the frame increases. Therefore if a practical solution is to be obtained methods of quickest convergence must be used. In problems involving stability this is even more important since the rate of convergence falls continuously with increasing load until at the critical load there will be no convergence at all. Hence normal application of Moment Distribution using modified stiffnesses and carry-over factors will only be practical for the simpler frames and the use of a more convergent method seems imperative. For this reason methods already used to improve the convergence in elastic analysis are used?

For the simplest problems the critical load can be obtained directly. For more difficult problems the critical load can be obtained by plotting a graph of the stiffness of the structure at various loads, and hence observing when the stiffness is zero. Alternatively and more easily the critical load can be obtained by testing the convergence of the calculation at a few values of the load parameter. When there is no convergence the critical load is indicated since a small applied moment could not be balanced by any finite rotations and sways.

Tests for Instability

There are two well known tests for stability of triangulated frames, the series criterion of Lundquist[3] and Hoff's[5] convergence criterion. Both are concerned with the rotation of one joint of a framework and in effect both are identical. In Lundquist's method a unit moment is applied at one joint, then all other joints are balanced and rebalanced in turn and the moment returned to the first joint by carry-over is obtained. If this moment is less than unity the frame is stable, if greater than unity the frame is unstable. In Hoff's method an arbitrary moment is applied to one joint and all the other joints balanced and rebalanced in turn. If the process converges to finite values then the frame is stable, otherwise it is unstable. In each case the testing procedure is applied at successively increasing loads until an unstable case is found. Both in fact are concerned with finding the load at which the joint under consideration has zero stiffness.

Exactly the same approach can be used to deal with sway problems, but in this case the testing distortion which should be used is not a simple joint rotation but a combination of joint rotation and sway. One particularly suitable distortion is called by the writer a " No Shear " operation.[7] This " No Shear " operation is taken to mean the rotation of a joint combined with just sufficient sway of the storeys above and below that joint to wipe out any horizontal constraints caused by the rotations alone.

It might be found for the elastic case that if a certain moment is applied at a joint by rotations and then the structure allowed to sway, the moment remaining at the joint is reduced. If axial forces are taken into account the reduction will be even greater, and as the axial forces are increased a load will be reached at which rotation of a joint and the subsequent sway necessary to balance horizontal constraints results in zero moment at the joint. Obviously the joint is now unstable since an infinitely small moment applied there could not be balanced by joint rotation. An alternative outlook is that at this load the frame could not resist an infinitely small horizontal force since such a force would cause out-of-balance moments at the joints which cannot now be balanced by finite rotations and sways.

In a typical frame however, the load at which the " No Shear " operation shows lack of convergence will not be reached. This would only happen if the joints above and below the one considered were fixed. If they rotate, moments will be carried-over to the joint considered, and it will be found that zero convergence will result from a combination of reduced stiffness of the joint considered and increased negative carry-over from adjacent joints. The example shown in Fig. 10 illustrates this effect, but to limit the labour required for the solution of such a problem it is advantageous to consider first an approximation to the critical load.

Approximations to the Critical Load

In most methods of obtaining critical loads, including those described in this paper, the amount of labour is greatly reduced if a reasonable approximation to the critical load is available. Such approximations are available for triangulated frames.[9] Combining operations of rotation and sway to build up a reasonably similar distortion to the critical mode enable such an approximation to be obtained for portal frames. In tall building frames for instance if the stanchions in one storey are relatively weaker than in any other the critical mode will consist of a large sway displacement for the weak storeys with smaller sways for adjacent lengths of stanchion. The remaining sways will be relatively small. Rotations will follow roughly the same pattern being large at the ends of the weak length of stanchion and smaller elsewhere. Hence a very good approximation to the critical load would be obtained if a block of three storeys were tested as shown in Fig. 7. However, the determination of the critical load of a block even only three storeys high is not quick.

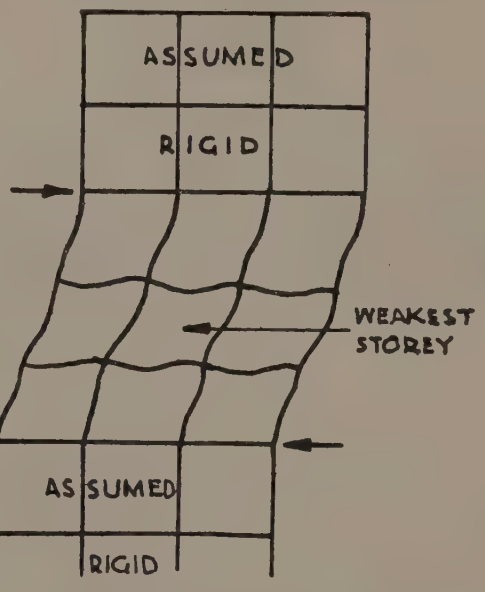


Fig. 7

A reasonable approximation to the critical load of this three storey block and hence to that of the whole frame can be obtained by further simplifying assumptions that each joint rotates the same amount in the critical mode and that the sways involved in the " No shear " rotations of the ends of all the stanchions in one storey are alike.

A test is made as follows : joints like *A*, *B*, *C*...*H* in the three storey block are given unit " no shear " rotations at some multiple of the design load and the sum of the moments appearing at the joints rotated is found. If this sum is zero for equal and like rotations of the joints then it can be argued that the frame must be unstable. At a load lower than the critical load the sum will have a large positive value, slightly above the critical load the sum will have a large negative value. In fact the sum of the moments appearing at the joints rotated is a sensitive indication of the stiffness of the frame, because the distortion itself approximates to the critical mode. It is important that the three storey block tested should include the weakest storey. If there is any doubt because of similarity of axial loads and beam stiffnesses the required number of tests should be made.

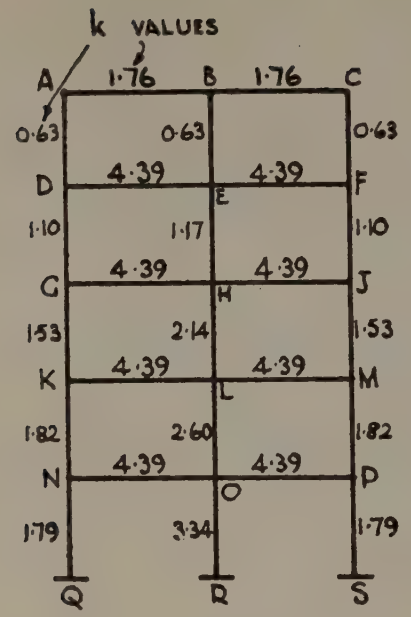


Fig. 8

In Fig. 8 is shown an example taken as typical of a modern careful design.[6] Joints *K, L, M, N, O,* and *P* are given unit " no shear " rotations and the sum of the moments appearing at *K, L, M, N, O,* and *P* is calculated. Two loadings are tested—seven times the design load and eight times the design load. The relevant information from Livesleys tabulation is :

Member	7 × Design Load			8 × Design Load		
	P/Pe	*n*	*o*	*P/Pe*	*n*	*o*
GK JM	0.455	—1.29		0.520	—1.89	
HL	0.651	—3.65		0.744	—5.88	
KN MP	0.536	—2.05	3.08	0.612	—3.01	3.87
LO	0.751	—6.11	6.69	0.858	—12.36	12.69
NQ PS	0.714	—5.01		0.816	—9.06	
OR	0.762	—6.50		0.870	—13.66	

Calculation

At a load of 7 *times the design load—*

Total moment at *K, L, M, N, O, P* =

$$\begin{aligned} & 8 \times 6 \times 4.39 \\ & + 2 \times 1.53 \times -1.29 \\ & + 2.14 \times -3.65 \\ & + 2 \times 1.82 \times -2.05 \\ & + 2.60 \times -6.11 \\ & + 2 \times 1.79 \times -5.01 \\ & + 3.34 \times -6.50 \\ & - 2 \times 2 \times 1.82 \times 3.08 \\ & - 2 \times 2.60 \times 6.69 \\ = & \ 78.9 \end{aligned}$$

At a load of 8 *times the design load—*

Total moment at *K, L, M, N, O, P* =

$$\begin{aligned} & 8 \times 6 \times 4.39 \\ & + 2 \times 1.53 \times -1.89 \\ & + 2.14 \times -5.88 \\ & + 2 \times 1.82 \times -3.01 \\ & + 2.60 \times -12.36 \\ & + 2 \times 1.79 \times -9.06 \\ & + 3.34 \times -13.66 \\ & - 2 \times 2 \times 1.82 \times 3.87 \\ & - 2 \times 2.60 \times 12.69 \\ = & -22.7 \end{aligned}$$

Linear interpolation gives the critical load as 7.78 design load.

It is possible to obtain an approximation by co sidering the rotations of joints at one level only, f instance *N, O* and *P* in the structure of Fig. 8. If ea of these joints is given a unit rotation and the tot moment appearing at *N, o* and *P* is calculated th this total becomes zero at the critical load. For a w designed frame in which the size of the members follo the loads to be carried this approximation will be fou to give a reasonable value when applied to the joi at the top of the lowest storey. For comparison t calculation has also been given :

Calculation

At a load of 7 *times the design load—*

Total moment at *N, O* and *P* =

$$\begin{aligned} & 4 \times 6 \times 4.39 \\ & + 2 \times 1.82 \times -2.05 \\ & + 2.60 \times -6.11 \\ & + 2 \times 1.79 \times -5.01 \\ & + 3.34 \times -6.48 \\ = & + 42.6 \end{aligned}$$

At a load of 8 *times the design load—*

Total moment at *N, O* and *P* =

$$\begin{aligned} & 4 \times 6 \times 4.39 \\ & + 2 \times 1.82 \times -3.01 \\ & + 2.60 \times -12.36 \\ & + 2 \times 1.79 \times -9.06 \\ & + 3.34 \times -13.66 \\ = & - 16.0 \end{aligned}$$

Linear interpolation gives the critical load as 7.72 the design load.

R. E. Bowles[8] has plotted the stiffness curve of t particular structure by calculating the stiffness various joints at different values of the load paramet He shows that the critical load is 7.30 times the desi load.

Special Cases—Symmetrical single bay frames with fixed bases—Fig. 9

1. *First approximation*—exact for one storey frame Rotate the two joints *A, A'* at the top of the weak stanchion length with a " no shear " rotation. T moment appearing at *A* is given by $n_1k_1+n_2k_2+6$ and the frame is unstable when this expression is ze

2. *Second approximation*—exact for two storey fram Rotate the two joints *A, A'* with " no shear " ro tions as before, and allow adjacent joints *B, B'* balance with " no shear " rotations.

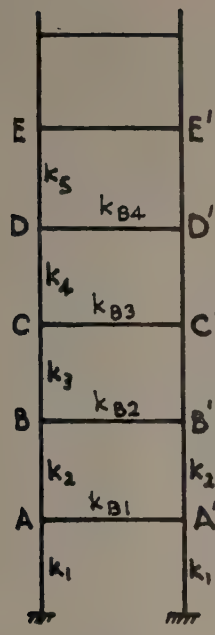


Fig. 9

For unit rotation of A, moment appearing at $B = -o_2k_2$

Therefore B rotates $+ \dfrac{o_2k_2}{n_2k_2+n_3k_3+6k_{B2}}$ to balance,

and the moment appearing at $A = n_1k_1+n_2k_2+6k_{B1} - \dfrac{(o_2k_2)^2}{n_2k_2+n_3k_3+6k_{B2}}$

Or writing $(n_1k_1+n_2k_2+6k_{B1})$ as S_A the frame is unstable $S_A - \dfrac{(o_2k_2)^2}{S_B} = 0.$

3. *Third approximation*—exact for three storey frames.

This process can be continued for any number of storeys. Thus for three adjacent joints if A is rotated and B and C balanced instability occurs when

$$\frac{S_A\, S_B\, S_C}{o_2k_2 \,.\, o_3k_3}\left[1 - \frac{(o_2k_2)^2}{S_A\, S_B} - \frac{(o_3k_3)^2}{S_B\, S_C}\right] = 0$$

This equation will give reasonable values when the frame is more than three storeys high.

4. For tall frames a reasonable approximation can be obtained by considering unit rotations of all joints. The moment appearing at a typical joint B is then $S_B - o_2k_2 - o_3k_3$, the last two terms representing the carry-over from the joints below and above B.

Thus at the critical load

$$S_B - o_2k_2 - o_3k_3 = 0$$

If the lowest joint is tested in this way the corresponding formula is

$$S_A - o_2k_2 = 0$$

since there is no carry-over from the fixed base. If the weakest joint is tested the approximations in this section will underestimate the critical load as the stiffening effect of adjacent joints is ignored.

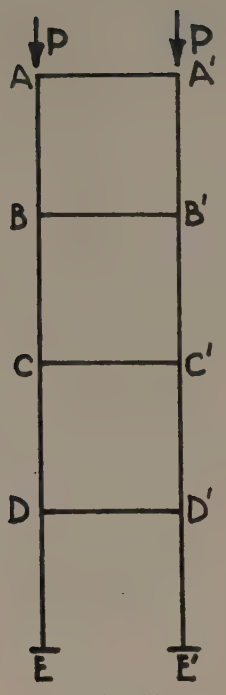


Fig. 10

Numerical Examples.

The frame shown in Fig. 10 will be used to demonstrate convergence and its approximate critical load is required. Since it is a four storey frame approximations 3 or 4 may be used.

Approximation 3 gives—

$$S_B = S_C = S_D = 6+2n$$
$$o_1k_1 = o_2k_2 = o_3k_3 = o$$

$$\therefore \frac{(6+2n)^3}{o^2}\left[1 - \frac{2\,o^2}{(6+2n)^2}\right] = 0$$

$$\therefore (6+2n) - \sqrt{2}\,o = 0$$

P/Pe	0.40	0.46	0.44
$2n$	—1.76	—2.67	—2.35
$-\sqrt{2o}$	—3.07	—3.56	—3.36
$6+2n-\sqrt{2o}$	+1.17	—0.23	+0.27

Hence critical load occurs when $P/Pe \simeq 0.45$.

Approximation 4 gives—

$$6+2n-2o = 0$$
$$\text{or } 3+n-o = 0$$

P/Pe	0.40	0.38
n	—0.88	—0.74
$-o$	—2.17	—2.07
$3+n-o$	—0.05	+0.19

Hence critical load occurs when $P/Pe \simeq 0.40$.

More exact determination of critical load

1. *Symmetrical single bay frames*

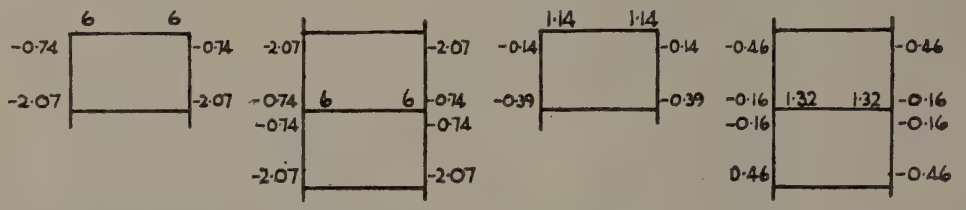
For the one storey case approximation (1) is exact and the expression $n_1k_1 + 6k_B = 0$ determines the critical load. Knowing the ratio $\frac{6k_B}{k_1}$ the critical load can be read off directly from the table. For example when $k_B = k_1$ the critical load occurs when $n_1 = -6$, i.e. $P/Pe \simeq 0.75$. A similar calculation can be carried out for two and three storey frames.

2. *Multi Storey Frames*

To obtain a more exact value of the critical load for a multi-storey symmetrical single bay frame, some particular multiple of the design load near the critical load is considered and the axial loads in all the members obtained. Using Livesley's tables the "no shear" patterns for the rotation of each pair of joints is obtained. From the inspection of these patterns it may be apparent whether the frame is stable or not at the particular load chosen. If the negative moment carried over to adjacent joints is in every case less than 50 per cent. of that balanced at the joint considered then the frame is almost certainly stable. If the negative moment carried over to adajcent joints is in every case greater than 50 per cent. then the frame is almost certainly unstable. Close to the critical load some of the carry-over factors will be less than 50 per cent. and some greater than 50 per cent.

In this case an arbitrary horizontal force F is applied at some joint on the frame and the sway moments obtained. Usually it will be best to apply the force at the top joint. The sway moments are proportional to $F \times h \times m$ where h is the storey height and m corresponds to the axial load in the stanchion considered.

$P/P_e = 0.38$



ROOF JOINT OTHER JOINTS
"NO SHEAR" PATTERNS USING LIVESLEY'S TABULATION

ROOF JOINT OTHER JOINTS
"NO SHEAR" PATTERNS REDUCED TO RELAX UNIT MOMENT

	E	D			C			B			A		TOTAL BALANCING MOMENT
		L	B	U	L	B	U	L	B	U	L	B	
DISTRIBUTION COEF.		·16	1·32	·16	·16	1·32	·16	·16	1·32	·16	·14	1·14	
ARBITRARY SWAY MOMENTS	-10	-10		-10	-10		-10	-10		-10	-10		
BALANCE	0	-3·2	26·4	-3·2	-3·2	26·4	-3·2	-3·2	26·4	-3·2	-1·4	11·4	70·0
CARRY-OVER	-9·2	0		-9·2	-9·2		-9·2	-9·2		-3·9	-9·2		
													49·9

Table 4a

$P/P_e = 0.42$

$n = -1.02$ $\quad O = 2.28$ $\quad$ C.O. FACTORS ~ ROOF JOINT $\frac{2.28}{6-1.02} = 0.46$
OTHER JOINTS $6\frac{2.28}{-2.04} = 0.58$

SWAY MOMENTS	-10	-20	-20	-20	-10	
BALANCE	0	20	20	20	10	70·0
CARRY OVER	-11·6	0 -11·6	-11·6 -11·6	-11·6 -4·6	-11·6	
BALANCE	0	11·6	23·2	16·2	11·6	62·6
CARRY OVER	-6·2	0 -13·4	-6·2 -9·4	-13·4 -5·3	-9·4	
BALANCE	0	13·4	15·6	18·7	9·4	57·1

Table 4b

$P/P_e = 0.44$ $n = -1.17$ $o = 2.39$ C.O.F. ~ ROOF JOINT $\frac{2.39}{6-1.17} = 0.50$ OTHER JOINTS $\frac{2.39}{6-2.34} = 0.66$

SWAY MOMENTS	-10		-20		-20		-20		-10	
BALANCE	0		20		20		20		10	70·0
CARRY OVER	-13·2	0		-13·2	-13·2	-13·2	-13·2	-5·0	-13·2	
BALANCE	0		13·2		26·4		18·2		13·2	71·0
CARRY OVER	-8·7	0		-17·4	-8·7	-12·0	-17·4	-6·6	-12·0	
BALANCE	0		17·4		20·7		24·0		12·0	74·1
CARRY OVER	-11·5	0		-13·7	-11·5	-15·8	-13·7	-6·0	-15·8	
BALANCE	0		13·7		27·3		19·7		15·8	76·5
CARRY OVER	-9·0	0		-18·0	-9·0	-13·0	-18·0	-7·9	-13·0	
BALANCE	0		18·0		22·0		25·9		13·0	78·9
CARRY OVER	-11·9	0		-14·5	-11·9	-17·1	-14·5	-6·5	-17·1	
BALANCE	0		14·5		29·0		21·0		17·1	81·6
CARRY OVER	-9·6	0		-19·2	-9·6	-18·9	-19·2	-8·5	-13·9	
BALANCE	0		19·2		23·5		27·7		13·9	84·3
CARRY OVER	-12·7	0		-15·5	-12·7	-18·3	-15·5	-7·0	-18·3	
BALANCE	0		15·5		31·0		22·5		18·3	87·3
CARRY OVER	-10·5	0		-20·5	-10·5	-14·8	-20·5	-9·1	-14·8	

Table 4c

$P/P_e = 0.50$ $n = -1.69$ $o = 2.79$ C.O.F. ~ ROOF JOINT $\frac{2.79}{6-1.69} = 0.65$ OTHER JOINTS $\frac{2.79}{6-3.38} = 1.06$

SWAY MOMENTS	-10		-20		-20		-20		-10	
BALANCE	0		20		20		20		10	70·0
CARRY-OVER	-21·2	0		-21·2	-21·2	-21·2	-21·2	-6·5	-21·2	

Table 4d

Using the " no shear " patterns an attempt is made to liquidate the sway moments. If the frame is far from instability the residual moment at every joint will be smaller than the original sway moment. If the frame is well beyond the stability limit every residual will be greater than the original sway moment. In either case no further calculation is required and the stability or instability of the structure is known after one cycle. The terms " far from instability " and " well beyond the stability limit " are relative. In practical cases this range may only cover the region from 90 per cent. to 110 per cent. of the critical load.

Close to the critical load some residuals will be greater and some less than the original sway moments. In this case a few cycles may be necessary to decide whether or not the frame is stable.

For regular frames the sum of the balancing moments required in each cycle will give an indication of the convergence or divergence of the calculations and is well worth tabulating.

Example

As an example of this kind of attack the critical load of the frame shown in Fig. 10 has been obtained. Since this example is used merely to demonstrate convergence without unnecessary complications, a somewhat artificial case has been chosen in which E, I and L are constant for each member, and the axial load in each stanchion length is the same, as is the " no shear " pattern for every joint except the roof joint.

To make clear the similarity between this procedure and rigid frame analysis Table 4(a) has been set out fully in the form that was used before[7]. In following tables work has been saved by considering the total moment at each joint. The approximations given above indicated a critical load between $P/Pe = 0.40$ and $P/Pe = 0.45$. To illustrate the points made about carry-over factors and convergence the stability of the frame has been checked at P/Pe values of 0.38, 0.42, 0.44, and 0.50 in Table 4.

Table 4(a) $P/Pe = 0.38$

The carry-over factors are seen to be 0.39 and 0.46 and thus the frame can be expected to be stable, in practice a bigger load would be tested immediately. After one cycle each residual is less than the original sway moment proving the frame is stable. The total balancing moment required in the second cycle is far less than that required in the first cycle.

Table 4(b) $P/Pe = 0.42$

The carry-over factors are 0.46 and 0.58 and hence the frame is close to its critical load. After one cycle joints A and C show a bigger residual moment than the original sway moment but B and D show smaller residuals. The total balancing moment required in the second cycle is less than that required in the first so the frame can be expected to be stable. This is proved at the start of the third cycle since all the residuals have been reduced below the original sway moments.

Table 4(c) $P/Pe = 0.44$

At this load the carry-over factors are 0.50 and 0.66 indicating that the frame is almost certainly unstable. After one cycle joints A and C show a bigger residual moment than the original sway moments, and B and D smaller residuals. However the total balancing moment required in the second cycle is greater than in the first, again indicating instability. The calculation shows all the residual moments greater than the original sway moments in the ninth cycle proving the frame

unstable. The total balancing moment required in any cycle is greater than that in the cycle before.

Table 4(d) $P/Pe = 0.50$

At this load the carry-over factors, and the size of the residuals after one cycle indicate instability.

From these calculations the critical load of the frame is seen to lie between $P/Pe = 0.42$ and $P/Pe = 0.44$ and these bounds are obviously close enough for design purposes.

Multi storey multi bay frames

Reference 7 shows a method of obtaining the " no shear " patterns for such frames in elastic analysis. In stability calculations the stanchion stiffnesses are of course modified and the appropriate values can be obtained from the tables of s, c, $s(1+c)$ and m; but otherwise exactly the same procedure is followed. When the " no shear " patterns have been obtained for some particular load the sway moments due to a horizontal force applied at the top joint are relaxed as in Table 4. If the calculations converge, the frame is stable and a greater load is tested. If the calculations do not converge the frame is unstable. The frame of Fig. 8 has been investigated in this way.

(*This paper was written in the Department of Civil Engineering, Liverpool University.*)

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Book Reviews

Publications of the International Association for Bridge and Structural Engineering (Vol. XIII, 1953, Vol. XIV, 1954) and Final Report of the 4th Congress, Cambridge and London, 1952. (Zurich : Leeman.) 366 pp., 9½ in. × 6½ in.). 318 pp. (9½ in. × 6½ in.). 528 pp., 9¾ in. × 7 in.).

These publications of the International Association for Bridge and Structural Engineering cover a large variety of subjects including both theory and practice and are amongst the foremost international series of original papers on structural engineering.

Vol. XIII contains nineteen papers, of which eight are in English, five in French, and six in German. Amongst the papers are the following : Open-Spandrel arch analysis, an experimental study of the relation between the properties of fresh and hardened concrete, beam on continuous elastic support, the theory of cylindrical shells, approximate formulae for the buckling of struts, contributions to the theory of plasticity, and beam vibrations due to a moving load.

Vol. XIV contains fourteen papers, seven in English, three in French, and four in German. The following are a few of the subjects treated : The web-buckling of flexurally loaded plate girders, the effect of variable repeated loads in building structures designed by the plastic theory, statistical calculation of strength of concrete beams, an experimental investigation of slabs subjected to concentrated loads, and the exact theory of prismatic structures formed by bending and its practical application.

The Final Report of the Fourth Congress is divided into three sections, the first dealing with safety of structures, loading, dynamic problems and deformation, and on the development of calculation. Metallic str tures are dealt with in the second section, and the th consists of papers on concrete and reinforced concr structures.

In all three volumes, the titles of the papers a summaries are given in the three languages.

Reinforced Concrete Arch Design (2nd Edition), G. P. Manning. (London : Pitman, 1954.) 8½ in. 5½ in. 192 pp. Price 30s.

This book, first published in 1933 and intended the use of designers, puts forward a definite met which, if followed step by step, will give an economi shape and thickness of arch in any given case. Val are given for influence lines for a complete range arches of different shapes and thicknesses, and th are fully worked out examples of the various type arch treated. Solutions are generally found by " slo deflection methods." Piled abutments and continu arches are treated by " displacement " methods a the flexible arch by an energy method.

The book contains much valuable data, particula for the bridge engineer.

Theory of Structures (4th Edition), by H. W. Coul (London : Pitman, 1955.) 8½ in. × 5½in. 564 xii pp. Price 25s.

A fourth edition of this well-known textbook been published to include new developments in the and design since 1950. Thus a new chapter on p stressed concrete design has been added, and some to-date references are given to new theories and meth which have appeared in journals. Some minor cor tions to the text have been made.

(*continued on page 244*)

Gasholder Development and Design*

Discussion on the Paper by Mr. W. R. Garrett, A.M.I.C.E. (Associate-Member of Council)

THE Chairman, introducing the lecturer, said as Constructional Engineer to the Northern Gas Board, whose territory extended from the North Riding of Yorkshire up to the Border and right across from the North Sea to the Irish Sea, Mr. Garrett had plenty of gas works on his hands, for the constructional work in which he was responsible.

MR. GARRETT then introduced his paper and showed a number of slides.

THE CHAIRMAN proposed a hearty vote of thanks to the author for his very interesting explanation of the work set out in his paper.

The vote of thanks was carried with acclamation.

Discussion

MR. H. KAYLOR (Associate-Member) said he wished to ask one or two questions which he hoped Mr. Garrett would not think too simple or elementary. First of all, in the case of the water seal for the bell, what happened if that water froze? Did it fix the bell and prevent its free movement? Taking the other extreme with regard to temperature, was it ever possible for evaporation to be so great as to break the seal?

With regard to the support of the steel tank, if the tank was supported at ground level and it was found that the ground needed to be piled, was it customary to pile the whole area to support the dead weight and the weight of the water, etc.?

As regards the welding, it could be seen from the slides that in order to make a proper seal for the bottom of the tank it was welded from the top surface and then tested after having been lifted from the ground, and then, as an extra margin of safety, it was welded on the under side. In the case of the spirally guided type of gas holder the welding of the wall plates was presumably done from the inside, but was there sufficient space outside to permit of the extra precaution in the same way?

Finally, the Author had mentioned that the neoprene seal sometimes tore: what was the normal life of that, and in what way was a leakage found if it did tear?

MR. GARRETT, in reply, said that the questions were not really as simple as Mr. Kaylor seemed to think. With regard to the water seal, in severe weather one had to avoid freezing by utilizing anti-freezing apparatus, which normally consisted of running a steam supply up to the cup and putting on an ejector so that the water was circulated with steam pressure. This kept the water on the move and liquid. There was another method which was used in the north but was not very popular in the south. The method was an electrical one and consisted merely of a Pyrotenax cable (a copper sheath filled with magnesium oxide and the three-core wires inside) that acted as a resistance like the resistance wire on, say, an electric blanket, and that was put on the inside of the cup in the water and it kept the cup water from freezing. Of course, if the cup water should freeze there was a possibility on thawing of a serious run-through of the shells to the bottom, but that was not allowed to happen.

On the question of evaporation it was normally considered that gasholders had to earn their living and therefore they had to go up and down; they were not required as ornaments, and every time the gasholder went into the tank it picked up a full cup of water, and that was the normal condition. Under abnormal conditions it might occur that a holder was fully inflated and it was desired to keep it fully inflated, and in that case the precaution was taken of putting on a water supply to make sure that the cups were always full of water; but that very seldom happened in this country because we did not get the hot weather to cause evaporation and make it necessary.

With regard to the support of the steel tank and the question of piles—with a steel tank it was possible to put in an ordinary reinforced concrete slab to carry it, or it could be piled. The question presumably referred to whether piles were put in round the periphery of the tank or whether it was piled over the entire area of the tank bottom. Actually if it was found necessary to use piles, it would be found as a rule that they had to be put over the whole area, the reason being that the bulk of the weight of the gasholder was in the water used in the tank. The weight contributed by the actual holder was comparatively insignificant. Again, over-turning from wind was surprisingly almost insignificant because, of course, it was on such a large diameter in relation to the height.

MR. H. KAYLOR asked if he could supplement his question. If the ground could normally carry one ton per square foot but the gas authorities insisted on piling, and piles were put in round the periphery, would it be practicable to put the ring of piles round the periphery and then introduce a flexible joint and keep the slab as a separate entity, in order to cheapen the cost?

MR. GARRETT, in reply, said it would be possible but he would be inclined to dome the inside and lift the plates up so that they were actually curved, and possibly put pre-stressing cables round the periphery of the slab, but he did not think that he would take the risk if the ground was of the nature that required piles. He should perhaps emphasise that he was not a person to put in piles if he did not consider that they were absolutely necessary.

With regard to the question on welding, there was sufficient space between the lifts. When the lifts were down, the minimum space was usually some 15 inches, and that was sufficient for the welder to get in although, of course, it was not very comfortable. External welding could be done when the holder was inflated under air. He definitely liked to weld both sides of the plate for the simple reason that if one had a lapped plate there was likely to be a build-up of corrosion between the two plates, which tended to burst the seam.

* *Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on the 13th January, 1955. Dr. S. B. Hamilton, M.Sc., B.Sc.(Eng.), A.R.C.S., M.I.C.E., (President) in the Chair. Published in "The Structural Engineer," Vol. XXXIII, No. 1, pp. 1-10. Jan. 1955.*

With regard to the neoprene seal there had not as yet been very much actual experience of this material in this country for gasholders. He had himself installed the first one in this country in 1950 and he anticipated that the neoprene fabric would last for 15 years, so he was unable to give the answer at the moment. He was afraid nobody else could supply the answer because the first of these holders was only some 10 years old, but the material was giving every indication of a reasonably long life.

On the question of detecting a leak, normally above the piston it was open to atmosphere and it should be as sweet as the atmosphere. If there was a leak inside, the gas would accumulate on the top of the piston and one would immediately know that something was wrong. It would be necessary to locate the leak quickly because one might get an explosive mixture above the piston. Air gas mixtures were very dangerous and they had to be attended to immediately.

Mr. Kaylor asked whether one got an indication of that from the register of the amount of gas being made.

Mr. Garrett, in reply, said that was not so and one merely relied on a sense of smell. It was possible to use indicators, but normally if one came out from the fresh air and went into the holder one knew immediately that there was something wrong if one could smell gas, and it had to be attended to immediately.

The Chairman asked whether it would be a matter of routine to patrol the holders.

Mr. Garrett, in reply, said that was one of the disadvantages of the M.A.N. and the Klonnë holders that they did require more supervision. It was really necessary in the case of the Klonnë and the M.A.N. to go into them at least once a day, and preferably twice. In the case of the flexible seal it was obviously not so important to have a daily inspection because one had a more or less mechanical seal, but at the same time he would not like to neglect the Wiggins holder for longer than about a week.

Mr. E. G. Robinson (Associate-Member) asked whether the Author had any experience of reinforced concrete tanks. Not very long ago the Institution had heard a very interesting paper by Mr. Turner about a reinforced concrete tank, but unfortunately he could not remember whether it had been put forward as a curiosity or as a practical proposition.

It was a great pleasure to see Mr. Garrett back in the south. He could well remember Mr. Garrett showing him a cathedral-like structure in the Old Kent Road which he had assured him was a gasholder, and having watched the riveters at work he could well believe that Mr. Garrett found it difficult to get men to do the job. They had been working in a 15 inch space and signalling to one another, and knocking in rivets like shelling peas. It had been a very remarkable and interesting pastime to watch them.

Mr. Garrett, in reply, said there were not many reinforced concrete tanks in the country. The only time one could possibly justify a reinforced concrete tank was when the tank was partially buried—in other words when it was not a practical proposition to put in a steel tank. One could, of course, always sink a steel tank into the ground and endeavour to protect it against the corrosive nature of the soil. It was a question of economics as to whether it would pay to excavate the ground, put in a concrete tank and utilize the excavated material as a bank round the outside. A tank built in that way had no real backing from t earth. The earth could not possibly be properly co solidated, so that to be on the safe side for desi purposes one had to put in heavy reinforcement, a it depended on how much one was prepared to all that backing to assist one, how much steel was us If a crack developed in the tank wall it was a ve costly and difficult job to seal, so that one rather err on the side of safety. To build a concrete tank co pletely above ground necessitated as much steel as th required for a steel tank, because one was dealing wi water pressure. Of course, if one went in for a p stressed concrete tank that could be an answer ; it w really a question of economics.

Mr. S. M. Reisser (Member) said the Author h spoken of the advantages of the M.A.N. and the Klon holders, but he had not mentioned the disadvantag He believed most of them had been put out of u during the war because there was a distinct war risk.

Secondly, he had been absolutely astonished to he that Mr. Garrett was still working on a joint efficien of 85 per cent. Who were the authorities that specif such efficiencies in 1955 ? He could mention a cou of dozen far more highly stressed structures. It wou also be interesting to see the Author's calculations the 92 per cent. efficiency in riveted work.

Mr. Garrett, in reply, said the disadvantages the M.A.N. holders was that one had to go ins pretty well every day to check that everything was order. There was a tar seal which, due to atmosphe changes, picked up moisture from the side sheets ins the shell, so that the tar which was circulated up the top had to have that water removed. It w possible during winter to have an ice layer round ins the polygon which could cause trouble with worki if one did not remove that water. The tar seal wa very important item : it had to be inspected, otherw there was the possibility of getting gas in the spa above the piston. Before the last war a terr explosion had occurred at Neunkirchen which unf tunately had been caused by work external to t holder. The men were working on pipe connections the holder and they ignited a connection carrying g that they were welding. The flame played on the s of the holder, buckled the side, the gas escaped throu the buckled plates above the piston, became mix with the air, there was an explosion, resulting in hea loss of life. It was quite true that the waterl holders in this country were put out of commiss during the war—all except one, which was the o method of storing gas in that particular place, oth wise that also would not have been used. There w the risk that one could get an explosive mixture abo the piston, but he believed it could be classified un present-day conditions as a justifiable risk, beca ideas in general had been changed on the subject risks and the proper inspection to avoid them.

With regard to joint efficiencies, he had built fo gasholder tanks in welded construction and he was v sorry to say that he had worked to a figure of 85 cent. Had he known then what he knew now would have lowered it considerably. It had cost awful lot of money to make good the defects in th particular holders in order to ensure that he was s with an 85 per cent. efficiency. There was no need remind the present audience of the two tanks at Faw which collapsed. Those tanks, he believed, w designed for 21,000 lbs. per square inch at 85 per ce efficiency : his own tanks, fortunately, were desig

or 7 tons per square inch at 85 per cent. but nevertheless, despite the lower working stress, he could not take the risk of not having every one of those seams above three-quarters thick radiographed. A radiograph cost anything from £1 to £1 10s. per foot, and the cost of doing that work had been high, and the anxiety had been worse. He was sorry to say he had found cracks in those tanks; they were definitely inclined to be notch brittle, and it might have resulted in those tanks splitting exactly the same as Fawley. Admittedly he had a little advantage over Fawley tanks because he had been working to a lower stress, but certainly with his present knowledge he would not increase that figure of 85 per cent. until he knew a lot more than he knew at present about notch brittle structures.

He was not suggesting, and he did not want anybody to be under a misapprehension, that riveted tanks were not subject to exactly the same thing. He had with him a report of a disaster which had occurred in 1912, and the date was rather significant—it was the 8th January. On the 8th January in that year the temperature indicated 7 degrees of frost. It was a riveted tank and it split from top to bottom, not through the rivet holes but through the plates—it was a notch brittle structure. The calculated factor of safety on the basis of 27 tons ultimate gave figures of 3.4 in the bottom tier, 3.59 in the next, and 4.0 in the next, and very much higher as one went up, and that tank split from top to bottom, so he did not wish to imply that riveted tanks were not prone to notch brittleness. If he wanted to work to a higher figure than 85 per cent. for joint efficiency of a welded tank then he wanted a better steel than he was at present getting. He could get a better steel which would cost him about £4 a ton, which then made a welded tank uneconomic compared with a riveted tank because there was nothing in the price factor to start with. He could get another steel costing about £14 a ton, and he was then safe to work to 85 per cent., possibly without radiographing; but if he was not prepared to go to £4 a ton extra then he was obviously not prepared to go to £14 a ton.

Mr. Reisser said there was to be a paper on brittle fracture in the near future, but he believed Mr. Garrett would find his 7 tons per square inch was very much the stress necessary to initiate a fracture. Also, if he was afraid of fractures was that not the answer to a previous question? Was that not a point in favour of reinforced concrete? Nobody had ever heard of brittle fracture in reinforced concrete as yet.

Mr. Garrett, in reply, said he had experienced the pleasure of building one reinforced concrete tank and he did not think he would like to build another. The tank was built as four segments leaving four wide construction joints. All bars were bonded at the construction joints and these joints were concreted last to make sure there was no movement of the four large segments. With every care and attention he still had not got a water-tight tank, and finally finished up by applying an asphalt lining to the interior of the tank. It might be possible under present-day conditions to build that tank in reinforced concrete and almost guarantee it, but he would not guarantee it in the north of England where his aggregates were generally very poor.

He did not wish to encroach on the future paper on notch brittle structures. He could say a great deal more about that subject but it would be unfair to enlarge too much on that particular matter. If he could possibly do so he would be attending that meeting and he hoped to make a contribution.

Mr. R. J. Fowler (Member) asked what controls insurance companies have on gasholder design and construction: have the insurance companies or the Gas Boards any structural specifications or design requirements? He was referring in particular to the spirally guided type: would there ever be a British Standards Specification on gasholders?

What is the accepted conception of wind loading of gasholders, and would it not be useful to take actual measurements on existing holders?

With regard to tanks, since the matter of notch brittleness was common to both welded and riveted tanks should not butt-welded vertical joints of the best quality be allowed, at least 90 per cent. efficiency, i.e., the efficiency of the riveted joints assuming correct plate material is used. His opinion is that butt-welded joints should be allowed one hundred per cent. efficiency as in normal structural design.

He believed it was right to accept that the circumferential joints were the more likely to be trigger-points of notch brittle plate fractures, and not the vertical joints, and he believed that records showed that notch brittle fractures had originated mainly, if not entirely, from circumferential joints. It was well known that notch brittle fracture could occur equally in thick plates riveted, that plates frequently cracked around the rivets and so could trigger off a brittle fracture.

The top curb in an inflated holder is in compression due to the radial tension from the crown sheets, and in a large holder that compression force might reach a total of 900 to 1,000 tons, which was not a load to take risks with. Was it good enough to design the top curb purely on cross-sectional area? He believed it should be considered as a ring in collapse under the effect of the radial inward tension, which might amount to 7 or 8 tons per foot run around the circumference. He also thought the centre of gravity of mass should lie on the line of the crown sheeting and not underneath it as in all the examples shown in the paper. When it was located underneath quite heavy torsional effects could be set up in the top curb and although it was not known whether such effects had ever caused failure of a holder they might quite well be the cause of many of the secondary troubles that could occur in that region. The main trusses on a large holder might meet the top curb at points 50 feet or more apart and the radial load they could withstand by virtue of their curved form as struts would be very small indeed, so that they were very uncertain in the support that they gave to the curb against collapse. It was similar to assuming that one was adequately supporting a very heavily loaded and long stanchion by strutting it with slender curved members at mid height.

A more superior arrangement of crown framing would be for the trusses to be substituted by true arch members thrusting against the top curb; thus when the holder was deflated the curb would be in tension by withstanding the arch thrusts instead of being idle. Thus considerable simplification of design would result.

Mr. Garrett, in reply, said there were no controls actually imposed by the insurance companies on the design and construction of gasholders, although they would control these items if that was required. He personally thought that gas engineers were not doing their duty if they did not design their own gasholders. They were very fortunate in that there were a lot of

good gasholder contractors who set a very high standard of design and these they were prepared to submit for the Gas Board's approval; but irrespective of that it was up to the engineer himself to make sure that he obtained a good gasholder.

As far as he knew, there were no limitations on the design stresses imposed on a gasholder. The only document that one had to be very careful about was the one of 1912 already referred to in answering a previous speaker, and the conclusions arrived at in that document were entirely wrong, but as the recommendations had never been rescinded he was afraid that one still ought to carry the gasholder carriage loads on the tank balcony with stiffeners down to the tank bottom in order to cover against possible trouble.

With regard to stresses, the gasholder designer was at liberty to use what stresses he liked, the same as other engineers. As regards a British Standards Specification for gasholders there was not one in this country; but the gasholder manufacturers had got together, as far as he knew—although they had not told him this—and they had more or less agreed their own standard designs between them. In Germany there was definitely a standard design of gasholder for any capacity one required.

There was no generally accepted conception of wind pressure on gasholders, or what figure should be taken. He personally believed it was usual to err on the high side and he would advocate not exceeding 32 lbs. per square foot, with reduction for shape, but most people considered that he sailed too near the wind and some contractors had even designed for 56 lbs. per square foot, which he thought was much too high.

He did not wish to be tied up with the paper which was to be given on notch brittle fractures, but Mr. Fowler had said, and quite rightly, that he would be inclined to go in for 100 per cent. for a fully prepared welded joint on the vertical seam. That meant that every one of those welds had to be radiographed in order to ensure that it was satisfactory, and also to start with one had to have a steel that would not be notch brittle at low temperatures. One would not get that with ordinary mild steel and one would have to adopt a special steel, and by that time it would pay one to lower the stress and not go for such an excessively high standard. As far as he knew the trigger points that had been found at the moment, and particular cognizance had been taken of those, were where the vertical weld met the horizontal weld. If one got, as one was very apt to get, slag inclusions or blow-holes, that was the point one had to be careful of, and he had taken particular care on some tanks that he was building to ensure that point being sound. He was not worried about the circumferential joint at all but he always tried to remove all trigger points in the vertical seams. He was still doubtful whether a double vee preparation of joint was the best form of construction for the circumferential joints. He was inclined to favour a lapped joint, with light fillet welds for the circumferential seams. He definitely believed it to be unwise to go to 100 per cent. joint efficiency with the present ordinary quality mild steel.

Mr. Fowler said that the point he had been trying to make was that with adequate welding with no defects and using notch ductile material one would be justified in stressing butt welds to 100 per cent. efficiency if, as at present, riveted joints on non-notch ductile material could be stressed to 90 per cent. efficiency. The present material is just as susceptible to notch brittle failure in riveted tanks as in welded.

Mr. Garrett, in reply, said there was one fundamental difference between a welded tank and a riveted tank. With welding it was possible to get a trigger point anywhere on the welds of the tank, but with riveting the holes were controlled : one had a plate and one put in a series of holes in that plate and they did not act as trigger points.

Mr. Reisser : Oh yes, they do.

Mr. Fowler : And they do crack around the rivet holes. You can get quite a star effect in them : I have seen it in boiler work and bridgework.

Mr. Garrett, in reply, said if they were not careful they would be getting on to the subject of notch brittle fractures again, but he hoped Mr. Fowler and Mr. Reisser would both attend that meeting.

He had missed out one point with regard to the efficiency of the riveted joints, namely that one could easily get 92, 92½ or 93 per cent. dependent upon the size of hole put in the plate.

With regard to crown framing he agreed that there could be a compression force in the top curb of 900 to 1,000 tons, dependent on the size of the gasholder, but he was not in agreement with the idea that the trussing played any part in restraining the shape of the curb or assisting the curb as a compression member. He agreed that it could do so but unfortunately when one came to erect holders the roof sheeting was invariably slack, and as the erector wanted to show how good he was, he merely tightened the main ties, lifted the truss and tightened the whole of the crown sheeting. The poor unsuspecting engineer, walking round, would say " that is a remarkably good crown—a beautiful shape —but the erector had in effect done the very thing that one did not want done on a gasholder because he had added a compression force in the top curb. Thus it should not be assumed that the trussing helped the curb.

Mr. Fowler : My argument was that it should not.

Mr. Garrett, in reply, said with regard to the design of the curb the only trouble was that he was disagreeing with something that had been done successfully for one hundred years and he had not got much ground to stand on, but the design of a curb as used today was fundamentally unsound. To expect an angle section with legs 3 foot or 4 foot to take a compression load of 1,000 tons was not reasonable : the proper form was a box, or something that would really take that compression load. The main difficulty was that gasholders had been built in that way for a long time and tradition died very hard. In addition the normally accepted design had been satisfactory. Whether it was possible to design a top curb more economically he did not know, but it would be a very brave man who appreciably increased the stress because, particularly in the design of a top curb, there were so many unknowns.

Mr. Fowler said, in considering the top curb as a compression ring and designing on its collapse one would find that the stress in the curb would come probably in the order of 3 to 4 tons per square inch according, of course, to how the plates were located but the plates could be arranged quite economically to give a stress of that value.

Mr. Garrett, in reply, said that was perfectly true. Gasholders had been built in that way for one hundred

ears and it would be a question of breaking new round. To make sure that everyone present appreiated the point that Mr. Fowler was making, the tresses involved in a top curb design were (a) the pull rom the crown sheets, (b) the weight of the side sheets, c) the pressure due to the external wind force and (d) . similar but smaller pressure due to the internal gas ressure pushing outwards—those were the forces and he load could vary for a small holder of, say, about ,000,000 cubic feet capacity, when it would be about 00 tons to well over 1,000 tons. It was a problem of lesigning a compression member to take that load. Ie agreed that present designs were not entirely satisactory, and he was not even satisfied with the calculaions; but he had not had an opportunity of really horoughly investigating the problem. It would need ome experimental justification if it were to be relesigned.

On the question of the help received from trusses when spaced at 50 feet apart he thought the top curb should never be designed for any assistance there, so hat automatically removed that point. It was a mistake to think that light trusses would hold in a circular compression member carrying a load of anyhing of from 200 to 1,000 tons.

He was afraid gasholder designing was like any other lesign : it was a question of economics through and through. The question of putting in a portal frame to support the top sheets looked very attractive at first sight, but on the whole it was very rarely done. If one could build the portal frame cheaper than a framing in the tank then the cheaper job was going to win.

Mr. J. A. Williams said in view of the very large area of thin plating in the average gasholder, could Mr. Garrett say how he specified painting, both exterior and interior, to prevent corrosion, and was it possible to say what was the expected life of a steel gasholder?

In connection with the erection of those tanks, and referring to Fig. 11 of the paper, there was very little sign of any temporary supports whilst something like eight strakes of thin plating to a large radius were being erected. It would be interesting to know what temporary works were in fact necessary, and also what sort of distortions one got in plating up and what tolerances were specified.

Finally, Mr. Garrett's second slide showed a column guided gasholder in which he said there was no diagonal bracing, which dated the holder because the engineers in those days had not realised the need for diagonal bracing. However, the tank seemed to be standing up quite well and he wondered what the moral was.

Mr. Garrett, in reply, said with regard to painting we had changed from the wrought iron era to the mild steel era. With wrought iron one could definitely, without any shadow of doubt and under really severe conditions, guarantee a life of about 80 years and, in fact, there were quite a number of cases where gasholders had served 100 years. In those days there was no special painting technique. Genuine ingredients were used in the paint, but the answer was probably in the material itself : the slag inclusions prevented corrosion penetrating right through. The building of mild steel gasholders was begun about 1880, the material being used first on the top curb, as might be expected because it was an expensive material at that time, and it was gradually used to a greater extent from that period onwards, and particularly from about 1900. As the puddlers who made the wrought iron sheets died out, so less and less wrought iron sheeting was available : if it could be obtained at a reasonable price today it would probably still be used except, of course, that the poor puddler only had about 3 cwt. of material on the end of his bar which meant that the sheets were very small as compared with the size used today. However, the industry had been forced into mild steel and for the painting of that he personally would never adopt anything other than phosphoric pickling at the works followed by a coat of red lead paint while the plates were still warm and then, after erection, any abraided portions should be touched up with red lead again. It should then be given another coat of red lead, and after that it did not very much matter : possibly an undercoat and a finishing coat could be used, because in his view the secret was to start off right. With regard to the inside, most of the gas boards in this country today debenzolised their gas so there was now little fear of the red lead peeling off, and he was quite satisfied with just the one coat of red lead inside, possibly touched up on the abraided portion and at the seams.

With regard to the erection shown in Fig. 11 he was afraid they did take chances and sometimes they came unstuck. He could remember a tank in course of erection collapsing not so very long ago—it was not one of his own—but those were contractors' risks. If the contractor liked to erect it without temporary support then he was taking a risk, but as far as the gas authority was concerned, providing the tank was perfectly all right and circular when it was finished, then they were quite happy. If one was erecting a holder that was welded, of course one had to work to very fine tolerances in the same way as with a riveted holder.

The moral of not having diagonal bracing was that about the period of 1870 quite a number of gas holders collapsed; When a gasholder was fully inflated it was in unstable equilibrium and if there was a lot of play, as so very often happened, in the guide rollers the whole bell could move. Quite a number of gas holders met their end by the mere fact that they were fully inflated, the wind blew the bell out of level so that the bottom curb was caught on the edge of the tank, and it finished up in the complete collapse of the holder. All gasholders in those days were designed by gas engineers—the contractors had no say in the matter then—and these failures made the gas engineers think that their designs were defective and hence they adopted bracing for the cast iron columns, which materially assisted in resisting the lateral thrust caused by wind pressure.

Mr. J. A. Williams asked whether, in the case of a welded holder, when one prepared the steel by pickling and then painting, it interfered with the welding?

Mr. Garrett, in reply, said he was afraid it did. It was necessary to leave a two inch strip and paint it with a special compound 'Ensis' fluid which did not interfere with the subsequent welding operations, and that was another nuisance with regard to welded gasholders. Of course, it was necessary to paint the rivet heads, but it was not so difficult. With a riveted holder one got a coat of paint in the seam, which one did not get with a welded holder, but with the latter one tried to seal both sides of the seam. That did not mean to say that one did not get moisture in between those seams, because they did not test the gasholder with the internal welding and then empty it and complete the external welding.

Mr. P. L. Capper (Member of Council) said the paper had shown that gasholders were more interesting from the structural engineering point of view than from their aesthetic appearance.

It would be interesting to know whether any of the local authorities or any of the Town and Country Planning Authorities were insisting on underground storage of gas for future construction. If so, was it essential from the economic point of view to use high pressure containers or was it economic in some cases to have an ordinary type of gasholder with the consequent very large amount of excavation necessary?

In the case of the high pressure holder it required a compressor plant to charge them up. It was perhaps rather outside the scope of the paper but he would be interested to know what means were used to reduce the pressure for supplying the ordinary mains. Was there any possibility of regenerative effect recovering some of the energy that had been put into it in compressing it?

MR. GARRETT, in reply, said the local authorities were having more to say every day on the question of gasholders and their aesthetic appearance, and it was rather unfortunate that they were more or less compelling the gas industry to adopt uneconomic designs. They were insisting that gasholders should be built as low as possible so that they did not cause such a blot on the landscape. Two holders might have the same capacity, but from a price point of view it was the holder of normal proportions which was more economic. Another effect was that if the gas authorities wanted a steel tank because the ground was suitable for that, the local authorities might try to insist on an underground tank so that the total height was less. If a gasholder cost more than it should do then it ultimately reflected in the price of the gas. Gas engineers were resisting as far as possible uneconomic gasholder designs and the question of who would win depended on how good the gas engineers were versus the Town and Country Planning officials, or how reasonable the latter were.

With regard to underground storage, his Board was just starting the first 10,000,000 cubic ft. capacity underground storage in this country, in the north of England. When all the industrial load practically ceased at about 5 o'clock on a Friday evening the gas would normally have to be restricted or alternatively burnt to waste at the coke ovens. With underground storage it was hoped to put into underground storage over the Friday night, Saturday and the Sunday surplus gas, and on the Monday morning the valves would be opened and the gas could be used during the ensuing week. That was the basic principl As regards the difficulties, there were quite a lot unknown factors but he believed it would be successfu It would take about fourteen months to get the ho prepared, about thirteen hundred feet below groun level, and he hoped when he returned to the north order the compressing machinery, but the compressic costs were really high. It would cost about 2d. therm, which might not sound quite so dear in th south, but the gas up in the north of England wa much cheaper. On the other hand, was it right to l that gas go to waste, or could it be justified on closin down a carbonising plant? There could be no re answer until one knew the actual cost that would b involved. It was possible to build any pressu cylinder one liked, above ground or below ground, bu it was again a question of economics. It was ver hard to convey an impression of the space occupied b 1,000,000 cubic feet of gas, but as possibly everyon present knew the Kennington Oval from its associatio with cricket, it might help to state that the large ga holder at the back of that ground held 5,000,000 cub feet of gas when fully inflated. When one starte talking in terms of pressure vessels to hold suc volumes the cost of storage vessels and compressin machinery became prohibitive. A pressure vessel use at Birmingham was 25 feet in diameter by 200 fe long, storing gas at 73 p.s.s.g., and only released ha a million cubic feet of gas. The cost of the vessel w really enormous and in addition one had to compre the gas, so unless it could be stored in the groun really high volume pressure vessels were not a practic proposition. Pressure vessels were normally a mea of feeding in from the tail end of a district so as to ev up the pressures in the whole of a system.

He was afraid as yet no means had been found recovering that potential energy stored in high pressu cylinders. There had been many bright ideas b nothing had come of them. Regulating the pressu coming out of the cylinder was quite easy; one simp put a governor and dropped the pressure down, b the potential energy was lost.

THE CHAIRMAN thanked Mr. Garrett for the ve entertaining and able way in which he had dealt wi the questions, and thanked the audience for attendi on such a very inclement evening.

The proceedings then terminated.

Book Reviews

(*continued from page 238*)

La Construction des Tunnels Galeries et Souterrains, by G. Bardout and P. Berny. (Paris : Eyrolles, 1954.) 6 in. × 10 in., 288 pp., 111 figures and diagrams. Price 3,400 fr.

All engineers and contractors who are concerned with tunnelling and other underground works will find this book most interesting and useful.

The work is very well presented and includes numerous illustrations covering methods of carrying out underground works, and showing how various difficulties can be overcome.

Data is given of interesting features which occur when one tunnel crosses over another tunnel and also when underground work has to be carried out agair adjoining buildings.

This book is particularly useful to contractors as gives a good description, with illustrations, of plant a various equipment used for tunnelling and undergrou works. The information given of the formwork for concrete tunnel lining is of great value.

Caissons form the subject of a particular chapt and the authors' extensive experience in the executi of the underground railways in Paris gives the varic examples described a tone of clarity and knowledge.

The book deals with both the practical and desi aspects of tunnelling and other underground works a also includes a chapter on costs of tunnelling.

P. J. G.

(*continued on page 254*)

Brittle Fracture Problems in Steel Construction*

Discussion on the Paper by Mr. G. M. Boyd, M.I.N.A., A.M.I.Struct.E.

THE PRESIDENT, introducing Mr. G. M. Boyd, said he was able to speak with considerable experience n brittle fracture problems in steel construction, subject which had been with us for a long time and hich had become of very considerable importance ithin recent years.

The meeting looked forward to an interesting discourse from Mr. Boyd.

MR. BOYD, presenting his paper, said he made no pologies for dealing with the subject of brittle fracture ecause he did feel that it was a problem of rather more nportance than was sometimes given to it.

Vote of Thanks

THE PRESIDENT proposed a very hearty vote of hanks to Mr. Boyd for his paper and for his able resentation of it.

(The vote of thanks was accorded with acclamation.)

Discussion

MR. DONOVAN H. LEE (Member) added his own ersonal thanks for an excellent paper, as printed in he Institution's Journal and as amplified by Mr. Boyd n his presentation of it.

Referring to the examples of fractures which were iven in the paper, some of which were well known, he aid that most of them were associated with welds, but lmost none of them were in the welds ; and he believed t would be agreed that that was nothing against weldng with normal quality mild steel. But the heating f certain steels could make them brittle near to the elds. Mr. Boyd had hinted that those steels were enerally defective, i.e. they had poor Izod values and hey were sensitive to temperatures, but he would like Mr. Boyd to have given data to identify steels unsuitble for welding. There might be sub-standard steels hich would not be sub-standard unless welded. We new, for example, that the Three Rivers Bridge, in anada, had collapsed on a very cold morning ; and fter the steel was examined an *Engineering News Record* editorial had pointed out that the steel comlied with the usual ASTM specification, but that it as of rimming quality, i.e. it was steel which was not ully killed. Fig. 12 showed the difference between ully killed steel and steel which was not fully killed.

Asking if the additions made to steel had any material effect he said that aluminium was added to elp kill the steel. Would Mr. Boyd explain whether ny particular accidental small traces of alloying or ther elements would affect either notch sensitivity or rittleness after welding ?

Again, what information could he offer concerning he effects of gaseous inclusions ? We knew, for xample (and Mr. Boyd had shown that clearly by one f his illustrations), the effect of improperly killed steel. lso he felt that hydrogen should not be overlooked, Nitrogen was present more in basic Bessemer steels. but did this affect weldability ?

His final question was whether any of the failures were linked with the use of bare welding electrodes.

MR. BOYD replied that it was quite true that the brittle fractures, although in many cases they started from a weld, did not as a rule follow the weld ; that led one to feel that, if welding had any effect on them at all, it facilitated the starting of fractures, but he could not imagine that it could have any great influence on the resistance to propagation of fractures through the parent plate. The effect of the weld was local.

Coming to the reference to defective and substandard steels, he said that those words ought to be taken with a great pinch of salt because, if the steels passed the specifications and tests, we had no right to say that they were either defective or sub-standard. Perhaps what was in Mr. Lee's mind was that steel which behaved in the manner indicated was behaving differently from what we usually expected of steel. Maybe our approach to specifications was not sufficiently detailed.

It was true that the steel for both the Duplessis and the Belgian bridge had passed the specifications.

He did not know of any evidence which connected bridge failures with the use of bare wire or any particular type of electrode. Indeed, the situation might be the other way round from that suggested by Mr. Lee, for he remembered a paper by Dr. Cotterell some time back in which it was pointed out that, at any rate from the point of view of hydrogen content, bare wire electrodes were the best. He doubted, however, that that would lead us to go back to their use.

The remainder of the questions put by Mr. Lee were of a metallurgical character. Explaining that he was no metallurgist, Mr. Boyd said he was happy and honoured that Dr. Allen, of the Metallurgical Department of the National Physical Laboratory, was present ; he invited Dr. Allen to come to his rescue in answering those questions, particularly on the effect of trace elements, gases and the merits of killed and unkilled steels.

DR. N. P. ALLEN said the question concerning the effects of various elements was a very difficult one to answer.

It was well to emphasise that there were two different kinds of elements having very different effects. Metals such as nickel, chromium and molybdenum had rather little effect on brittle fracture, although he would be a bold man who would say that they had no effect. But there were certain other elements—oxygen, hydrogen, phosphorus, nitrogen, carbon and probably many others —which had very important effects ; they could alter the temperature at which the mild steel changed from tough to brittle by 100°C. or more. The effects of these elements depended very much, in a complicated way, on the heat-treatment the steel had received ; their effects had a tendency to depend on the amounts of the other elements which were present. For example, the effect of oxygen was not independent of the amounts of carbon and phosphorus present. The subject would need to be studied a great deal more before we reached

*Paper read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1 on the 27th January, 1955. Dr. S. B. Hamilton, M.Sc., B.Sc.(Eng.), A.R.C.S., M.I.C.E. (President) in the Chair. Published in "The Structural Engineer," Vol. XXXIII, No. 1, pp. 11-19 (January, 1955).

any finality, but it was clear that it would be very difficult to control the tendency of steel to brittle fracture by specifying its composition.

That, continued Dr. Allen, emphasised the importance of the point made by Mr. Boyd about having satisfactory tests by which to assess resistance to brittle fracture. He took the opportunity to urge that the subject was of immense importance and was one on which both metallurgists and engineers must be prepared to change their ideas fundamentally. The steelmakers had usually considered mild steel to be a material on which a test for toughness was not necessary; they had held that all ordinary mild steel was adequately tough for its purpose, and that notched bar impact tests were not required. But notched bar impact tests on mild steels did show which would go brittle at moderately high temperature and which would remain tough at low temperature, and that being so, the steelmakers must accept sooner or later some test for that quality. It must be realised that a steel structure could bear an entirely different order of stress safely if it were above the temperature at which the steel changed from the tough to the brittle behaviour than if it were below that temperature; if it were safely above that temperature as shown by an adequate test, it was reasonable to assume that stress of the order of the yield point could be applied. But if the temperature of the steel was below the transition temperature, the stress which could safely be applied was only about a quarter of the yield point.

Engineers had to decide whether they were prepared to use steel below its transition temperature and accept a low level of stress, or would insist on having a steel which was known to be above its transition temperature and adopt a higher level of stress.

Mr. E. Asquith, speaking as one who had suffered from the occurrence of brittle fracture in two large storage tanks, endorsed Dr. Allen's comments concerning tests and the difficulty of getting the steelmakers to carry out the tests. They were all very intent on production, but the users would like them to put a little more energy into the carrying out of tests to ensure that their steel was to the right specification and of the quality which was necessary to give security of the structures in which they were used. He very much appreciated also Mr. Boyd's comments on the types of tests which he felt should be carried out—the Charpy Impact test, and so on—and invited his comments on what was meant by " P.403."

Referring again to the two failures he had mentioned, Mr. Asquith said it was felt that the prime cause was defective welding, which had initiated the brittle fractures, that if the welding had been perfect the fractures would not have occurred. He could confirm from first-hand experience that the brittle fractures did not follow the welds; they might be initiated at the welds, but were propagated through the parent metal. He asked for Mr. Boyd's comments particularly on the Charpy Impact test, and also on carbon and manganese contents, which he felt might be critical.

Mr. Boyd said the question of the types of tests to use was one to which anybody concerned with the problem of brittle fracture was devoting a lot of time and thought; so that he would express only a personal opinion. We were mainly concerned with the resistance which the material would have to the propagation of a fracture. The resistance to initiation was useful, but it was very easily broken down by treatment such as welding; so that we were concerned mainly with resistance to propagation. That being so, it was quite clear that we must look at the types of fractures whic occurred in specimens. We should have specime large enough to display the characteristic brittle fra ture, and there were about half a dozen tests availab which could be applied. There was the slow notche bend test proposed by Van der Veen; the Robertso test; the Navy tear test; and the Tipper notch tensi test. Any of them would inform us about propagatio

The Charpy test was very widely used, but he w not very clear about how it should be interpreted. H did know that the energy, considered by itself, was n a good guide; we must consider not only the energ but also the percentage of crystallinity in the fractur It was interesting that when Charpy had invented t test his idea had been to break a piece of steel with hammer and to examine the fracture; and as I Orowan had said some years ago, our troubles on started when we began to measure the energy in t Charpy test.

P.403 was the number that was given to one Lloyd's Rules, which stated that the steel to be use in certain parts of a welded structure must be special approved for that purpose. There were 40 or mo steels which had been approved under that Rul Therefore, any steel which had been approved b Lloyd's Register of Shipping under that particul Rule was for brevity termed a " P.403 steel," meanin that Lloyd's Register considered it to be sufficient good to be used in welded ship's structures, and o could rely on it being a good deal better than one wou get if one left the matter to chance. All the " sup notch tough "steels, or most of them, complied wi the Rule, but not all " P.403 " steels could be calle " super notch tough."

Lt.-Colonel G. W. Kirkland, M.B.E. (Member, an Hon. Curator), said the problem discussed in the pap had been with us for many years, but it had be brought very much to the fore in our minds as a resu of the recent " Comet " aircraft tests.

He had been interested in Dr. Allen's remarks, an asked if Mr. Boyd could give an indication of the ran of transition temperatures for mild steel complying wi B.S.15.

Mr. Boyd said the answer to the question wou depend on how one defined " transition temperature If one defined it on the basis of Charpy V-notch ener one obtained one value, and on the basis of fractu appearance one obtained another. The ordinary r of B.15 steels might have Charpy 15 ft. lbs. transiti temperatures anywhere between —40° and +40°. If one judged on fracture appearance it might va from —20° to + 30°C.

Mr. S. M. Reisser (Member) recalled that at a rece meeting of the Institution the question of brittle fractu had cropped up, but it had been agreed to postpone t argument until Mr. Boyd presented his paper. So people apparently thought that the occurrence of brittle fracture could be prevented by reducing t efficiency of a welded joint to 85 per cent., i.e. using working stress of some 7½ tons per square inch, and asked whether Mr. Boyd considered that such a pro dure would eliminate the risk or whether it mer resulted in a waste of steel. One of the graphs in t paper appeared to indicate that a brittle fracture cou occur at a stress of only 4 or 5 tons per square inch a Dr. Allen had mentioned circumstances in wh stresses of only some quarter of the yield stress co be used with safety, but he would be grateful if M. Boyd would clarify the point still further.

The term "Transition Temperature" had been defined, but the value varied with the type of test used and the term as such was really meaningless until a standard test had been agreed to.

Mr. Donovan Lee had suggested that welding temperature might have something to do with the occurrence of brittle fracture, and he wondered whether Mr. Boyd could say anything about a recent failure of this type in a continental mill building. He had been told that the frame in question was of riveted design and the thickness was little over ½ inch.

Could Mr. Boyd also say whether there were any records of brittle fractures having occurred in any of the special notch tough steels which had been mentioned.

Also, he wondered whether Mr. Boyd had any records of similar failures in reinforced concrete.

Finally, Mr. Reisser wished to compliment Mr. Boyd on his very fine work in this field and the many excellent papers which he had read on the subject.

Mr. Boyd said Mr. Reisser's question about lowering the stress enabled him to say that in his opinion, taking a practical view of the whole thing, it would not be sensible to attempt to design structures to work below their transition points, as defined by Dr. Allen. It would mean that we should have to work to stresses which were ridiculously low. But there was the point that even below the transition temperature as defined, the steel did not necessarily fail even if it had higher stress, and indeed it often withstood quite high stresses. The operative word in Dr. Allen's contribution, however, was "safely." There was always considerable risk, and he could not help feeling that we must face the situation which had been put quite bluntly by Dr. Allen.

As to what was the transition temperature we had really to make up our minds, and again Dr. Allen had helped him by emphasising the dilemma. We should have to make a decision very soon.

With regard to Mr. Reisser's question concerning fracture which had occurred at a steel mill on the Continent, he said he had heard about it but he had no details.

Also he had no details about fractures in reinforcing bars, but he had heard rumours; one of them concerned a prestressed concrete pipeline. However, he knew even less about reinforced concrete than about metallurgy.

He had no knowledge of any brittle fractures occurring in the steels which were claimed to be specially notch tough, but the use of them, of course, was comparatively small. In steels complying with Rule P.403—and very careful records had been kept ever since the Rule had been in operation—no fractures had been recorded as having occurred in service. A few short cracks had occurred, however, during construction.

Mr. R. J. Fowler (Member) added his personal thanks to Mr. Boyd for a very excellent paper.

Asking if Mr. Boyd had had experience of brittle fracture in a rolled steel section, he said that most of the fractures described in literature on the subject seemed to be in fabricated plate; but, of course, a very large percentage of structural work was composed of rolled steel sections. He recalled an instance where two rolled steel joists, 12 in. by 6 in., were tied together toe-to-toe to form a buck stay, and cranked at a small angle about the Y-Y axis. To perform the cranking, half of each flange was gas cut; the cranking was completed and re-welded, but the welding was done by saddle welding round the surfaces of the mating pieces. The buck stay, 30 ft. long, was handled on site in cold weather; it had received a small bump when dragged along the ground and had snapped completely in two right across the sections, even through the unwelded parts.

Another type of fracture he had experienced had occurred in a type of boiler. There was a ring composed of 1¼ in. plates 10 in. deep, about 6 or 7 ft. in diameter, horizontal, and it was coated on its inside face with a Stellite material to serve as a surface to resist abrasion. The ring was welded to the base of the vertical circular boiler and in parts the ring reached temperatures of the order of 600°F., whereas the steel in the boiler to which it was welded reached probably just over 212°F. The boiler operated only at low pressure. A crack had developed in the ring right through its thickness during the operation of the boiler, and on the outside of the ring a whole series of Leuder lines had appeared, showing that the ring had yielded in tension. It did not fail with a loud report, but just grew and was probably initiated by a sort of craze cracking on the Stellite coating on the inside surface.

In order to make a check an ordinary tensile specimen was made of boiler quality steel, one face of which was coated with Stellite, and it was tested in tension. It had failed at 23 tons/sq. in., the elongation was 5 per cent. on 8 in., and the yield point just did not show up in testing. The specimen had failed, of course, with a loud report, because it was tested at room temperature.

Mr. Fowler asked if those two examples could be related to brittle fracture.

Mr. Boyd replied that he had records of a number of failures in rolled steel sections. One rather spectacular case which had had a lot of publicity and was still on show in the University of Liege, Belgium, was a rolled steel joist which had split from end to end, except for about 4 in., when it was unloaded from wagons in cool weather. Of course, there had been quite a number of cases where joists had cracked across when they had fallen on to something. The bent section mentioned by Mr. Fowler, where a crank was put in by chopping a piece out of it, was a notched bar specimen, and to doctor it up by welding was to ask for trouble. He suggested that that was a kind of practice to be very severely avoided.

The other cases mentioned were very interesting, particularly the cracking of the thick plate at high temperature. He asked if the parts had actually fractured at the operating temperature of 600°F.

Mr. Fowler replied that they had so fractured.

Mr. Boyd said that certainly it was a brittle fracture, but that temperature was rather outside the range of his experience of that kind of thing. In ordinary mild steel made to ordinary specifications he had seen very brittle fractures at temperatures of something like 140°F., which were quite high enough. But he rather felt that to put a hard coating of Stellite on to any steel would give the ideal notch effect and, if there were any tendency at all to brittleness, that would bring it out.

Mr. B. L. Clark (Associate-Member) commented that in the course of such discussions there seemed always to be nearly a riot as between welding and riveting. With regard to brittle fracture in ships, we had been building ships for many years and figures showed that there had been about 400 failures of welded ships. One felt that there must be some analogy that could be drawn. Before ships were welded there had

been failures in riveted ships and some of those ships had to be towed back to port ; but there were not so many failures as had occurred since we built welded ships during the war.

To his mind a pre-fabricated Bailey bridge, a welded light structure, was a type of construction which provided a fair test for brittle fracture. Such bridges were subjected to very low temperatures ; they were in use all over the world at temperatures very much below zero, but he did not know of any which had fractured, although there was a lot of welding in them. Those structures carried very heavy loads in bending, working stresses were up to 12 or 14 tons/sq. in., and when they had failed it had been due to overload, imposed by heavy traffic or to miscalculation on the part of an engineer.

Asking if Mr. Boyd had any records of fractures in Bailey bridges, Mr. Clark said he knew that a lot of trouble had been experienced in the early days of fabricating those bridges, there having been cracking due to welding ; but that trouble was cured after careful consideration of the problem involved.

The behaviour of ordinary railway lines was very interesting and a number of fractures occurred in them during cold weather, but there was no welding. Railway gangs had told him that just a nick on one side with a " Jim Crow " during cold weather would break them easily ; in hot weather there could be quite a bow in them and very often they would not break.

Could a very high percentage of silicon in steel make a difference ? He did know that plates, in manufacture, were likely to get considerably higher silicon contents than ordinary rolled steel sections.

Finally, he asked what effect shock had on the occurrence of brittle fracture. If the results were plotted, did they follow the same sort of curve as (say) a transition curve, or was the material not subject to fracture as the result of shock ?

Mr. Boyd said that, so far as American welded and riveted ships were concerned, he was not quite sure what Mr. Clark was getting at, or whether he thought riveting was better than welding, or vice versa. But it was stated in the paper that brittle fracture was by no means confined to welded structures. There was one riveted ship which had the remarkable record of having broken in two twice.

In regard to Bailey bridges he was out of his depth. But he would say that either those bridges had been made with a satisfactory type of steel or they had been very fortunate. He believed that the sections used were rather light, a fact which probably helped ; there was quite a definite trend towards greater liability as the thickness increased.

The behaviour of railway rails was always a fascinating subject ; they seemed to stand up to their job all right, and yet if they were subjected to an Izod or Charpy impact test at room temperature the result would be 5 or 6 ft. lbs. That was, generally speaking, one of the anomalies, and it seemed to be one of those cases where the Charpy value should be interpreted according to the type of steel used.

He had no evidence on which to be able to say that high silicon content, considered by itself, was good or bad, and he hesitated to ask Dr. Allen to discuss it. Generally speaking, the silicon contents we were concerned with in ordinary structural steels were in the range from about 0.05 to about 0.30 per cent., and in that range it did not seem to make a lot of difference one way or the other, except that one could tell very roughly what type of steel one was using by reference to the silicon content.

About shock, as he had stated in the paper, it was able to facilitate the commencement of fractures, but he did not think it had any influence at all on the character of the fracture which occurred after it had got under way.

Mr. Clark said that apparently he had not made himself clear when he had referred to shipping. The point was that about 400 welded ships had fractured. He did not mean that welding was the only cause of fracture, but he could not quite see why so many welded ships had suffered in that way, and presumably not the same number of riveted ships.

Mr. Boyd replied that the welded ships were built mostly under the American emergency programme, and they were all alike ; there were quite a number of failures, and there might be a statistical element in that. But it seemed quite clear that the presence of welding did something to the steel which rendered it more liable to the initiation of fractures, and he believed one of the biggest challenges to welding today was to find out what that effect was and how to cure it.

Mr. F. V. M. Bell (Chairman, Wales and Monmouthshire Branch), asked if Mr. Boyd would care to deal with the significance of the thickness of the material in relation to liability to fracture ; and he asked if it were true that there had been no liability to brittle fracture in material less than $\frac{1}{2}$ in. thick.

Mr. Boyd agreed that there was certainly a thickness effect ; the thicker the material, the more liable it was to suffer fracture. There were more fractures, on the *pro rata* basis, in steel $1\frac{1}{2}$ in. thick than in steel $\frac{3}{4}$ in. thick or something of that kind ; but fractures had occurred in material having a thickness of $\frac{1}{2}$ in. and even less. One did not know at the moment whether the effect was a geometrical or a metallurgical one, but he was rather inclined to the view that it was a metallurgical one. Maybe the faster the material could cool from the rolling temperature, the better—a thick plate did not cool so fast as a thinner one, and this would affect the grain size.

Mr. W. Barr, in a written contribution, stated :

On page 17 the Author states that there is no simple method of correlating the results of one test with those of another. The writer will admit that there is no exact correlation, but there is a vast amount of evidence to show that there is a sufficient measure of correlation for most practical purposes insofar that all tests put steels into a similar order of merit.

For instance as far as structural steels in current production in Britain are concerned, those steels shown to have exceptional toughness by the Charpy Vee-Notch test have their superior toughness confirmed by other tests. This knowledge has been applied in the development of steels with super-notch ductility which, judging by the demand for them, seem to be giving a satisfactory account of themselves in service.

It is to be regretted, therefore, that in a paper dealing with general aspects of the brittle fracture problem the Author has presented the data shown in Fig. 12 which is misleading, at least insofar as British steels are concerned. This graph, relating energy absorption to fracture appearance in the Charpy Vee-Notch test, shows steels having high energy absorption values associated with predominantly cleavage fracture. With British steels this is the exception rather than the rule.

In a paper to be published in the near future dealing with the correlation of the Charpy Vee and the Van Der Veen Slow Bend tests, it is shown that the energy absorption and fracture appearance criteria give the same general picture of the relative notch ductility of a range of steels. Isolated exceptions have been found, but while these are of great interest to the theoreticians they should not be permitted to cloud the practical issue of providing steels with adequate notch ductility for current engineering requirements.

Exception can also be taken to the second part of Fig. 12 purporting to show the relation between silicon content and the percentage cleavage in the fracture of Charpy Vee-Notch specimens broken at 0°C. Taken out of a proper context, this graph is quite misleading. It may be pointed out that three of the exceptionally notch tough structural steels at present on the market in Britain have silicon contents of approximately .05 per cent., .15 per cent., and .3 per cent. respectively.

Apart from this unfortunate lapse, the Author has presented an interesting and concise statement of the subject which will amply repay study by structural engineers.

Mr. Boyd said that so far as the correlation of the results of one test with those of another was concerned, the operative words in Mr. Barr's statement were to distinguish between ordinary steels and those of exceptional toughness. There was no doubt that the kind of steel he was thinking of was in a class completely different from that of the ordinary run of steels and it did not matter very much what test was used to distinguish it from them. But the purpose of the tests was to enable us to sort out the unacceptable and the acceptable steels from the ordinary run of mild steels.

About Fig. 12 being misleading, he did not know which way it led us, but he did know that it reveals the facts in a perfectly straightforward manner. Mr. Barr was probably thinking wholly in terms of British steels which might show as a group on the curve; Mr. Boyd would not say on which part of the curve they would be, but the points shown related to structural steels from all over the world. It was said that the lower part of Fig. 12 was even more misleading than the top part; but again it just gave the facts. He did not really know what they meant; but if the silicon content were an index of the degree of de-oxidation of the steel, then it meant that de-oxidation by itself was not a good guide to crystallinity.

Incidentally, the lower part of Fig. 12 was included because of a criticism levelled at the upper part. It was said that when there was an anomalous effects it was all a question of silicon; he had plotted the silicon contents to see if there were a connection, but had found none.

However, he was very grateful to have Mr. Barr's reactions, which were always interesting.

Mr. O. Bondy (Member) said he had particularly enjoyed reading the paper and listening to the discussion.

On page 12 it was stated that to most engineers a fracture was just a fracture. That was so; but he wondered if it were true, as was also stated, that if there had been more attempts to distinguish between the many kinds of fractures the true causes might be more accurately assessed, and their repetition avoided. A number of structures had failed and there were probably more reports on failures than any of us could digest, but still we had not arrived at reliable conclusions. If we had done so we should be in a position to suggest how to avoid brittle failures in the future. He was not in a position to forecast, but he wondered if Mr. Boyd could do so.

On page 17 it was mentioned that only a very small shock might be required to start a fracture. Was not that really the basis of the practice of hammering a pressure vessel during hydraulic tests in order to show up any tendency to cracking?

Looking at the paper from the structural engineer's point of view, continued Mr. Bondy, it had occurred to him that many structural engineers felt rather lost in what might be a metallurgist's paradise. He wished that the metallurgists had shown us a definite way in which to avoid brittle fractures in future. He did not remember seeing any definition of that except in the recent publication† by the well known W. S. Pellini, of the United States, who claimed—and one would like Mr. Boyd's reaction to it—that there was no chance of brittle fracture initiation if the Charpy V-notch impact value was more than 10 ft. lbs., and that there was no chance of propagation of such a fracture if this value was more than 20 ft. lbs., always related to the particular ambient temperature, and with the reservation that that claim referred only to rimming and semi-killed steels. If metallurgists would agree to bring down the problem to such simple terms as Pellini had tried to do, structural engineers would be in a much happier position than at present.

Mr. Boyd said he was very grateful to his old friend Mr. Bondy, whose questions were extremely pertinent, as all the others had been.

On the interesting question as to whether the recognition of types of fractures would help us to avoid them in the future, he said he did not know to what extent it would help, but he was very confident that if we did not pay attention to this aspect of the problem we should reach no conclusion at all. However, he felt that we were making progress in the matter at long last and that before long we should have something very much more definite.

If there were a tendency to fracture, the hammer test would bring it out. Whether it was good or bad he did not know, but it would seem to be wise to avoid the liability to cracking before making a pressure vessel.

About engineers and metallurgists, he agreed with Mr. Bondy's remarks. It was borne in upon him more and more as time progressed that engineers would have to learn a little about metallurgy; they could not rely too implicitly upon anyone.

Having read of Pellini's work with very great interest, he said he believed Mr. Bondy was quite correct so far as the facts were concerned; we must realise the implication that what was said would hold for rimmed and semi-skilled steels. It meant that before one started to apply the Charpy test one must find out what steel one was testing; in other words, the point that stood out from Pellini's work was that the Charpy values given related only to particular steels.

Mr. Donovan H. Lee (Member) said he felt that Mr. Boyd was at something of a disadvantage, in view of Mr. Barr's letter, and that he had not noticed that in fact the second graph in Fig. 12 showed quite clearly that silicon did not have much effect. He had had some correspondence with Mr. Barr some time ago and he had a great respect for Mr. Barr's knowledge, as he was sure we all had; but there appeared to be something wrong in the letter. But Mr. Barr had

(†) Welding Research Supplement (U.S.A.), Sept. and Oct., 1954,

mentioned something about killed and semi-killed steel. Mr. Lee qualified that by saying—and he did not know whether or not Dr. Allen would agree—that the oxygen content did matter a lot and hydrogen (and possibly the nitrogen content) mattered even more. Silicon would help to kill the steel ; but there were other ways of killing steel, such as by the use of aluminium.

Mr. Boyd said he too was puzzled by Mr. Barr's statement that the lower part of Fig. 12 was misleading. The graph simply showed, as Mr. Lee remembered, that silicon had little effect. He thought Mr. Barr must have something else in mind which he had not made clear.

Mr. W. Heigh (Chairman of the Scottish Branch) said he had enjoyed the paper very much. A point which worried him a little was that we knew we should for ever be improving steel and improving welding conditions in the workshop and on site ; all the time very important structures were being made by welding and the amount of trouble was really very small. Further, if we were to blame welding for initiating cracks, probably the fault lay with the engineers themselves in that they did not insist that the welding was done properly. Good welding was no more expensive than bad welding ; the factors which caused cracks were all well known and they should not be there, and we should see that the welding was good.

Again, did the material crack under other conditions ? Of course it did, it always had done. He had had experience for many years with riveted and bolted work, and cracks had occurred due to bending. In that connection he recalled an experience in the early days of gas cutting. A man was controlling the torch by hand—the machine was not automatic—and was running it along the centre of the web of a beam of the order of 12 in. by 6 in., and about 40 ft. long. When it reached 5 ft. from the end, the beam parted from end to end. The condition obtaining might be called a comfortable heat condition, the spread of the temperature round about was quite good, so that the trouble was not expected, and the steel was of reasonable type.

As a rule we expected mild steels to be very ductile and to have a fairly good notch impact value. Mr. Heigh felt that we should be very much more optimistic about the facts. In nearly all the examples of failure he had seen, the conditions had been rather trying ; there had been in the actual design and in the service which the structures would have to meet a condition which caused yielding, and which one might call notch effect. There were cases in which a whole beam suddenly became immensely weaker in hogging, for example.

As Mr. Boyd had said, engineers must not depend on the metallurgist all the time, but should co-operate with him and try to study his job, just as they always liked him to study theirs ; and they should not blame bad welds on to anyone but themselves if they had not ensured that the welds were properly made, for the fault was really their own.

Mr. Boyd was most grateful for Mr. Heigh's remarks. He hoped he had not given any impression that in his view engineers must not trust metallurgists. What he had meant to convey was that they should not rely implicitly on the metallurgist and blame him when anything went wrong ; it was a case for collaboration, and each man must learn a little of the job and the problems of the other.

It was true that the proportion of trouble that aros was quite small, and that was one of the reasons wh he was an optimist, because he believed that a ver small average improvement in the steels that we ha today would get us " out of the red " ; but at th moment the risk was too great to accept.

A very graphic example of flame cutting leading t fracture had been given by Mr. Heigh. Mr. Boyd w reminded of a story his uncle had told him about a occasion in a shipyard on the Continent, when th platers were so clever that they had made a few chis marks in the plate and then hit it with a hamme whereupon it fractured right along ; No doubt, h commented, that had saved quite a lot of gas !

Mr. G. Coates asked if Mr. Boyd would agree th the different tests mentioned would place steels in th same order of merit, and also that no small scale tes could represent service conditions; if so it would n matter which test were adopted.

It was disturbing to hear Mr. Boyd say that wel could initiate fracture, but did not necessarily affe the propagation of cracks. There were instances welds stopping the propagation of cracks.

It had been suggested that brittle fracture troubl had come with the welding era, but we had now al the thick plate era and the trouble might be due to combination of geometrical and metallurgical facto which would also apply to riveted structures.

Mr. Boyd commented that Mr. Coates was ove modest in asking some of his questions, for he w probably capable of answering them himself.

He believed that if the steels tested were ve different the different tests would place them in t same order of merit, but if they were nearly alike t different tests would place them in a different order merit. Probably a statement made in Americ literature some time ago had led to the belief that, matter what test was used, one would get the requir result. But he was being forced more and more to t conclusion that it did matter very much what type test was used. For example, if one wanted to devel a steel which would produce high Charpy energy valu one would do one thing, and if one wanted steel of go crystallinity value one would do something different.

He agreed very much with Mr. Coates that a lot our troubles might be due to the fact that we we using thicker and thicker plates ; and it was very mu on the cards that, if we were to build riveted shi today with steel of the kind of thickness with whi we are now building welded ships, we should be fac with trouble.

The President, at the conclusion of the discussio expressed thanks to Mr. Boyd for the very pleasant a able manner in which he had answered the questio raised.

Written Discussion

Mr. J. Ritchie (Associate Member) writes :

May I offer my congratulations to Mr. G. M. Bo on the excellent paper on " Brittle Fracture Probl in Steel Construction " which he read before t Institution on the 27th January, 1955, and for the m interesting discussion which followed.

Might I venture to ask if Mr. Boyd would care favour me with a reply to the following points :

1. Is continental Bessemer steel more susceptible brittle fracture than British steel to BS.15 ? If con nental steel must be used should punching be avoi

id drilling of all holes specified? Alternatively, is ere anything to be gained in reverting to the practice under-sized punching with reaming to give the quired hole diameter?

2. Has Mr. Boyd any reason to believe that hot-pped galvanising may be a factor in accentuating the anger of brittle fracture, particularly where punched oles have been used?

3. Would Mr. Boyd subscribe to the recommenda-ons of A.S.T.M. "Recommended Practice for Safe-uarding against Embrittlement of Hot Galvanised tructural Steel Products and Procedure for Detecting mbrittlement"?

4. Is there any evidence to lead one to suppose that rittle fracture is more likely to occur in high tensile teel to BS.548 than in mild steel to BS.15?

I am moved to ask these questions because of a recent ailure in a $3\frac{1}{2}$ in. by $3\frac{1}{2}$ in. by $\frac{5}{16}$ in. galvanised angle ection of Belgian Bessemer steel with punched holes; he failure occurred across a punched section, also one n a second member of $2\frac{1}{2}$ in. by $2\frac{1}{2}$ in. by $\frac{1}{4}$ in. section vhich had an identification notch punched into the toe f the angle. The failure in this latter case occurred cross the line of the notch.

These two sections are, I think, much smaller in size han any of the members discussed at the meeting; nd the fact that failure can occur in such small cantlings may be of general interest.

Reply to Mr. Ritchie.

Mr. Boyd writes:

There is a general impression that Bessemer steel is more susceptible to brittle fracture than open hearth steel, but there are wide variations in steels produced by both methods. Any generalisation of this kind is difficult to justify.

Punching is severe treatment for any steel subject to strain-ageing, and Bessemer steel is usually more subject to this than open hearth steel, owing to its usually higher nitrogen content. Punching under-size and reamering is preferable to punching alone, particularly when the hole diameter is small compared with the thickness.

I have no experience on the effects of hot-dip galvanizing.

The liability to brittle fracture bears little relation to tensile strength, so it would not be possible to generalise in answer to question No. 4. If the added strength is obtained simply by increasing the carbon content, one would expect increased liability to brittleness, but this may be offset by other features, such as alloying elements and heat-treatment.

The fracture of such small sections is certainly interesting, and it would be useful to ascertain to what extent it was due to inherent brittleness of the material before punching or galvanizing.

Institution Notices and Proceedings

IMPORTANT NOTICE TO SUBSCRIBERS

Subscribers, Newsagents and Subscription Agencies at home and abroad are asked to note that as from the 1st January, 1955, copies of THE STRUCTURAL ENGINEER can only be obtained on application to the Secretary of the Institution of Structural Engineers, 11, Upper Belgrave Street, London, S.W.1.

Subscription Rate

The Annual Subscription, post free, to any part of the world is 44s. 6d.

Single Copies

The price of single copies is 3s. 6d. or 3s. 9d. if sent by post.

In order to fulfil all demands, subscribers and others are requested to state their requirements as soon as possible.

The usual rate of discount will apply.

HONOURS AND AWARDS

In offering their sincere congratulations to the following members on the distinctions recently conferred upon them, the Council feel they are also expressing the good wishes of the Institution:

Order of the Bath, Military Division—G.C.B.
Admiral The Earl Mountbatten of Burma (Honorary Member).

Order of the British Empire—O.B.E.
Dr. S. B. Hamilton (President).

EXAMINATIONS—JANUARY, 1956

The Examinations of the Institution will next be held at centres in the United Kingdom and Overseas on January 10th and 11th, 1956 (Graduateship) and January 12th and 13th (Associate-Membership).

REPRESENTATION

The Council have appointed the following Institution representatives:—

Codes of Practice Committee for Buildings:

Drafting Committee, Steel and Aluminium Corrugated Sheet Metal Roofing:
Mr. H. A. Cadwell (Member).

Drafting Committee, Flues for Larger Appliances in Buildings (Installations over 100,000 B.T.U. per solid, liquid and gaseous fuels):
Mr. E. P. Featherstone (Associate-Member).

Drafting Committee, Roof and Yard Drainage:
Mr. C. E. Cannons (Member).

Regional Advisory Council for Higher Technological Education, London and Home Counties:

Regional Advisory Committee for Civil Engineering (including Structural):
Mr. L. E. Kent (Vice-President)—re-appointed for three years.

REPRESENTATION OVERSEAS

The Council have appointed Mr. R. A. Sutcliffe (Member) to act as the Institution's Representative in East Africa, in place of Mr. R. C. Marc.

ENGINEERING MARINE AND WELDING EXHIBITION

The Directors of the Engineering, Marine and Welding Exhibition, which will be held at Olympia from the 1st to 15th September, 1955, have invited all members of the Institution to visit the Exhibition on Tuesday, 13th September, 1955. A ticket of admission, available for two persons, which may be used for the official visit or on any other single day, may be obtained on application to the Secretary.

DRURY MEDAL AWARD

The fifth competition for the above award will take place in 1955. The subject is the design of a mobile crane.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary : envelopes to be marked in the top left-hand corner, " Drury Medal Award."

The closing date for the competition is October 1st, 1955.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say, primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution, or failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate in the presence of a Justice of the Peace or a Commissioner for Oaths.

RESEARCH AWARDS

The Council have instituted a Research Prize Fund, from which awards may be made annually to the author or joint authors of papers describing original research which they have carried out. Research awards may be made for papers read at Headquarters or in the Branches and published in the Journal, or for papers published in the Journal only without being read at an open meeting.

The assessment for such awards will be made annually, but awards will be made only to the contributors of such papers as reach a standard judged by the Literature Committee to be satisfactory.

Work submitted under this scheme must be original and may include any of the following :

(a) investigations of an experimental or analytical character ;

(b) studies of historical or statistical records ;

(c) improvements in principles or methods of construction ;

(d) research into methods of structural engineering and building, the nature and use of plant and the organisation of engineering work ;

(e) any related or combined studies which are deemed by the Literature Committee to be of a research character.

In cases where the research work described in the paper was not the work of one individual, the names of all the collaborators should be given in the paper.

Awards may take any or all of the following forms : A research medal ; a diploma ; a money prize.

Application for consideration for a research awa must be made to the Secretary of the Institution, a in preparing papers for reproduction in the Journ authors must comply with the conditions laid down f all such contributions. Particulars of these conditio may be obtained from the Secretary.

In judging research papers, the following factors w be considered :

(a) the nature of the subject and its conclusions ;

(b) the value of the paper in advancing the scien and art of structural engineering ;

(c) the standard of preparation and orderly arrang ment of the subject-matter.

Research papers will also be eligible for adjudicati for the Institution Medals if they comply with t Regulations governing those awards.

The closing date for the receipt of applications respect of papers published in the Journal betwe October, 1954, and September, 1955, is October 3 1955.

MACLACHLAN LECTURE COMPETITION, 195

The closing date for the receipt of entries for the ne Maclachlan Lecture Competition is Friday, March 30 1956. The general conditions of the competition are follows :

1. The Institution of Structural Engineers sh institute a written lecture to be known as the Maclachl Lecture and to be held annually.

2. The subject of the Lecture may be on any aspe of Structural Engineering as long as in every seco year the subject shall be confined to steel structur

3. Entrance into the competition for the Lectu shall be confined to Graduates and Associate Memb of the Institution who are under the age of 32 years.

4. All papers entered for the competition shall submitted to assessors to be appointed by the Coun of the Institution, and all such papers (including t prize-winning Lecture) shall be available for publicati in the Journal of the Institution at the discretion the Council.

5. No paper submitted shall have been published read elsewhere.

6. The winner of the competition shall be requir to present the Lecture to a meeting of the Institut at which he will be presented with the sum of £17 1

7. Should a competitor's paper be considered wort of ranking second in merit he will receive a consolati award of £5.

8. In the event of there being no winner of the co petition in any one or more years, whether because lecture submitted is considered to be of sufficient me to warrant award, or for any other reason, the Insti tion shall transfer the above sums to the Research Fu of the Institution.

PARTICULARS OF THE COMPETITION FOR 1956

1. The Maclachlan Lecture will be given at a meet of the Institution to be arranged towards the end 1956.

2. The subject of the Lecture shall be on any asp of structural engineering.

3. The work should be submitted as the script o lecture which the author, if successful in the co petition, will deliver before an audience in the cours about one hour. The development of mathemati

rmulæ and detailed calculations should be avoided as r as possible in the text; if they are essential they ould be embodied in appendices. Photographs, awings, graphs, etc., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be own as slides, a list of these should be included.

4. Six copies of each Lecture should be submitted nd should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the nstitution is Friday, March 30th, 1956.

BRANCH NOTICES

LONDON GRADUATES' AND STUDENTS' SECTION

on. Secretary : D. E. Capelin, 40, Thornton Crescent, Old Coulsdon, Surrey.

LANCASHIRE AND CHESHIRE BRANCH

oint Hon. Secretaries : J. L. Robinson, A.M.I.Struct.E., 314, Northenden Road, Sale, Manchester; M. D. Woods, 58, Spring Gardens, Salford, 6.

MIDLAND COUNTIES BRANCH

The Annual General Meeting of the Branch was held n Birmingham on the 22nd April, 1955, when the llowing Honorary Officers and Committee were lected for the Session 1955-6 :

hairman : Mr. W. D. Christie.
ice-Chairmen : Mr. R. J. Fowler and Mr. L. J. Griffiths.
lon. Treasurer : Mr. W. Phillips.
lon. Auditors : Mr. R. H. Howl and Mr. M. D. Picknell.
lon. Secretary : Mr. L. A. Firminger, 656, Chester Road, Erdington, Birmingham, 23.
lon. Assistant Secretary : Mr. J. R. Chaffer.
lcting Hon. Assistant Secretary (*Derby District*) : Mr. J. A. Jackson.
lcting Hon. Assistant Secretary (*Nottingham District*) : Mr. C. Probert.
ommittee : Messrs. L. P. B. Arthur, G. Botteley, H. Button, G. F. Eley, J. E. C. Farebrother, H. C. Griffiths, S. Willis, J. C. Billington, H. V. Hill, W. J. N. Mayo, I. A. Morton, R. D. Teague.
x-officio : Messrs. G. S. McDonald (Vice-President), A. P. Mason (Member of Council), H. V. Hill (Associate-Member of Council) and all Past Chairmen who are members of the Branch.

MIDLAND COUNTIES BRANCH GRADUATES' AND STUDENTS' SECTION

The following Honorary Officers and Committee Members have been elected for the Session 1955-6 :

hairman: Mr. G. F. Wilson.
lon. Secretary : Mr. J. E. Judas.
ommittee : Messrs. W. R. Hall, F. W. Boylan, M. R. H. Thomas, F. J. Pardoe, J. Neal.

NORTHERN COUNTIES BRANCH

lon. Secretary : O. Lithgow, 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

lon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., "Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

lon. Secretary : W. Basil Scott, M.I.Struct.E., 19, Waterloo Street, Glasgow, C.2.

SOUTH WESTERN COUNTIES BRANCH

The Annual General Meeting of the Branch was held at Plymouth on the 27th May, when the following Honorary Officers and Committee were elected for the Session 1955-6 :

Chairman : Lt.-Col. R. Hazzledine, O.B.E., T.D.
Vice-Chairman : Mr. E. W. Howells.
Hon. Auditors : Mr. H. J. Scoles (Past Chairman) and Mr. J. C. Peters.
Joint Hon. Secretaries : Mr. E. W. Howells, 10/12, Market Street, Torquay, Devon; Mr. C. J. Woodrow, J.P., "Elstow," Hartley Park Villas, Tavistock, Road, Plymouth.
Hon. Treasurer: Mr. E. W. Howells.
Committee : Lt.-Col. R. J. Dean, Messrs. H. J. Scoles, H. W. Miller, F. W. Potter, F. J. Powell, M.B.E. H. Toft, W. C. Tyler, F. M. Upson, L. F. Vanston (Past Chairman).

WALES AND MONMOUTHSHIRE BRANCH

Hon. Secretary : K. J. Stewart, A.M.I.C.E., A.M.I. Struct.E., 15, Glanmor Road, Swansea, Glam.

WESTERN COUNTIES BRANCH

The following Honorary Officers and Committee have been elected for the Session 1955-6 :

Chairman : Mr. N. G. T. Ball.
Vice-Chairman : Lt.-Col. E. Ward, T.D.
Hon. Secretary : Mr. E. Hughes, 23, Southdown Road, Westbury-on-Trym, Bristol, 9.
Hon. Treasurer : Mr. F. K. Fennell.
Past Chairmen : Mr. G. B. R. Pimm, Professor J. F. Baker, O.B.E., Messrs. C. H. Williams, C. J. D. Boxall, P. C. Girdlestone, Professor A. G. Pugsley, O.B.E., Mr. E. N. Underwood.
Committee : Messrs. J. E. Collins, F. A. Long, J. W. Lorraine, G. C. Mander, C. E. Saunders, R. H. Barnett, C. J. D. Boxall, A. J. Brownston, F. G. Clarke, C. W. H. Lawrence, Professor A. G. Pugsley, O.B.E. (ex-officio), Mr. C. H. Williams.

YORKSHIRE BRANCH

The following Honorary Officers and Committee have been elected for the Session 1955-6 :

Chairman : Professor R. H. Evans.
Senior Vice-Chairman : Mr. J. P. Robinson.
Junior Vice-Chairman : Mr. A. L. Percy.
Immediate Past Chairman : Mr. Leslie Preston.
Hon. Auditors : Mr. C. C. Begg and Major H. C. Taylor.
Hon. Secretary : Mr. E. Wrigley, 17, The Drive, Alwoodley, Leeds.
Committee : Messrs. T. Barlow, John Dossor (Past Chairman), R. H. Dowson, W. Eastwood, W. N. Espie, J. Jenkins, E. Lightfoot, I. MacGregor, F. A. Maddock, A. Robb (Past Chairman), W. Hunter Rose (Past Chairman), Major H. C. Taylor, L. C. Walker, Captain F. A. Whitaker, Mr. D. R. S. Wilson (Past Chairman).
Ex-Officio : Professor J. Husband (Past Chairman).

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box No. 3306, Johannesburg, South Africa.

During weekdays Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. Phone 34-1111, Ext. 257.

Natal Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co. Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary: R. Stubbs, M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

Book Reviews

(*continued from page 244*)

Surveying and Levelling for Students, by Bernard H. Knight, D.Sc., Ph.D. (London : Contractors Record, 1954.) 152 pp., 8½ in. × 5½ in. Price 18s.

This is a book of about 150 pages and provides a good reference book for students preparing for the Land Surveying section in the various Institution examinations. The method of approach to surveying operations is clear and assumes no previous knowledge of the subject.

The subject matter is well illustrated by sketches and photographs and sufficiently well chosen exercises are included for practice purposes.

The nine chapters cover chain and traverse surveying, levelling, setting out, triangulation and tacheometric surveying. All the information is up-to-date and useful reference is made to the many new types of surveying instruments which are now in use.

Quite rightly, the book mkes no reference to the very old instruments which are seldom used in present-day practice.

For students and engineers, architects, surveyors and builders this book is recommended as an excellent book of reference.

N. C. S.

Data Book for Civil Engineers (Vol. III. Field Practice, 2nd Edition), by E. E. Seelye. (New York : Wiley, 1954 ; London : Chapman & Hall.) 8 in. × 5 in. 394 plus xvii pp. Price 120s.

This is the second edition of a book aimed at furnishing the field engineer or inspector with the basic data required when he is cut off from other references, and to provide him with a degree of background knowledge. All the tables and other data have been brought up-to-date and new material has been added. The section on air-entrainment has been greatly extended and there is a special reference section on surveying, and a short course on soil mechanics. It is a useful and well-produced book, and is of a convenient size for carrying about.

The Manufacture of Iron and Steel, by D. J. O. Brandt. (London : English Universities Press, 1953.) 8½ in. × 5½ in. 384 pp. Price 15s.

This book on the manufacture of iron and steel has been written with the full co-operation of the British Iron and Steel Federation and the City and Guilds of London Institute, and is mainly designed for operatives and students in the industry, who will find it a valuable textbook. Special features of the book are the short forewords placed at the beginning of each section to give a general introduction to a new subject, and the three revision chapters which attempt to build up by means of diagrams a picture of the interrelationship of the facts learned and how the various sections of the industry depend upon one another. The book forms a good introductory course to all interested in the subject.

Engineering Metallurgy, by Dr. E. M. H. Li (Distributed by Cleaver-Hume Press, Ltd., Londo published in Philips' Technical Library, 1954.) 246 p 6 in. × 9 in., with 170 illustrations and 56 tabl Price 32s. 6d.

It requires only a glance at the table of contents f this treatise on metals to appreciate the value this bo will be to the metallurgist and engineer alike.

The opening chapter, devoted as it is mainly mechanical properties of metals and alloys togeth with the various tests employed, contains informati much sought after by students and others, and wh would only be expected to be found in a book seve times this size. Of the six remaining very fine chapte two in particular—(*a*) Alloys of Iron with Carbon a Other Metals, and (*b*) Non-Ferrous Metals and Alloys- are outstanding, by virtue of the very wide range subject matter dealt with and presented, as it primarily for the needs of and in a form acceptable the designer and practising engineer.

Of nearly two hundred exceptionally neat diagra over half are devoted to graphs together with the me structures in diagrammatic form. When to all this added fifty-six tables of technical data, one will ha a fair idea of the wealth of information available. T only criticism that might be offered in this connect is that much of the data is given in the metric syste

The book, clearly printed and rounded off with a really good index, cannot fail to find a place on the d of all interested in the study and use of metals and th alloys, whether in colleges, foundries, or in the numer industries.

W. S. S

"The Ironmonger" Pocket Book of Tables (Edition). (London, 1954. 6 in. × 3½ in. 56 Price 3s. 6d.

Fully revised and containing much additio information, this new edition contains over a hund tables, dealing amongst other subjects with iron, s and non-ferrous metal sheets, sections, tubes and w gauges, bolts and nuts, tanks and cylinders. All ta based on British Standards have been brought up- date according to the latest Specifications.

The Modular Catalogue (Vol. I). (London : Modular Society Ltd., 1955.) 11 in. × 8½ in. Fre charge to subscribers.

This first issue of the Modular Catalogue consist introductory material, ten sheets of products an loose-leaf binder for the first volume comprising at one hundred sheets. These will be issued serially f time to time, and not until all the sheets of the volume have been issued to members, probably, by end of the year, will the catalogue be available to public. Each sheet of the Catalogue is devoted modular product of one manufacturer which is a able generally, the four inch basic module being basis of the catalogue.

The Load Bearing Capacity of Metal Structures*

By H. V. Hill, M.Sc., A.M.I.C.E. (Associate-Member of Council)

Introduction

The subject of this paper is an extremely wide one since it embraces a large portion of the whole field of structural engineering. It is, therefore, impossible to deal with all aspects of it in the space available. Certain parts of the subject can only be touched on briefly and the Author proposes to deal with it mostly in general terms.

Whilst one should not make it too habitual to reflect over the past it is often extremely beneficial to consider past history and its influence on the present and future. With this in view it is proposed at the outset to look backwards into our subject and to examine a few interesting factors which have had a great deal of influence on the development and growth of our profession.

The Relationship between Theory and Practice

Developments in theories regarding the behaviour of structures have often been quite separate from practical considerations. Whilst the early mathematicians interested themselves chiefly in problems of elasticity the early experimenters were chiefly concerned with the ultimate strengths of their structural components. Thus at the beginning of the 18th century the French school (Ecole Polythechnic) was largely working on mathematical problems.[1] Elasticians, from this school, such as Navier, St. Venant, Poisson, Laplace, Lagrange Lamé, Clapeyron and Cauchy were evolving theories. In like manner, later Thomas Young, J. Clerk Maxwell and others were working in this country on the mathematical conceptions of elasticity.[2]

Practising English engineers in the early part of the 18th century, on the other hand, were not very interested in the mathematical side of engineering. For example, it is said of Telford that "he had a singular distaste for mathematical studies and never even made himself acquainted with the elements of geometry." Telford, as is well known, was the first President of the I.C.E. (1821) and remained President all his life. He was, therefore, very influential in the engineering world of his time.

Of the later engineers, Fairbairn (Tubular Bridge over River Conway and Britannia Bridge over Menai Straits) seemed to rely largely on tests on models, and when asked to calculate the capacity and deflection of tubular girders had to call in his friend, Hodgkinson, a mathematician who had a much greater theoretical background to assist him.

The knowledge of some of these early applied mathematicians was, however, surprisingly advanced, and Hodgkinson is reported to have said about tubular girder construction: "It appeared evident to me, however, that any conclusions deduced from received principles with respect to the strength of thin tubes, could only be approximation; for these tubes usually give way by the top or compressed side becoming wrinkled, and unable to offer resistance, long before the parts subjected to tension are strained to the utmost they would bear . . ." He, therefore, proposed that models should be made for testing purposes but it is interesting to note his reference to wrinkling of thin walled sections. (Fig. 1.)

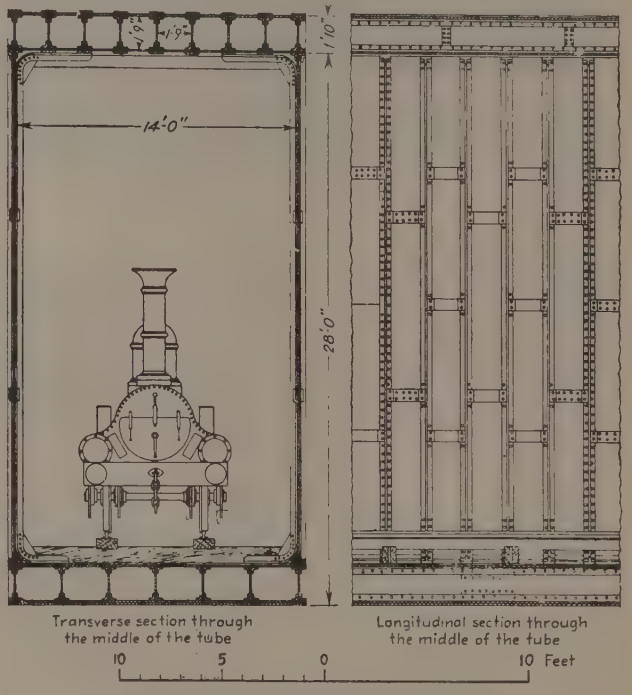
14'-0"
28'-0"
Transverse section through the middle of the tube
Longitudinal section through the middle of the tube
10 5 0 10 Feet

Fig. 1

Fairbairn's method of testing had to be used but Hodgkinson was able to compute the deflections within 20 per cent. of test values.

The probability is that Prof. W. J. M. Rankine was first in Britain to attempt to relate theory and practice in engineering education. His classic book entitled "Manual of Civil Engineering" (1861) was an amazing treatise as also his other publications on the theory of earth pressures. All his work was of immense importance and much is still of great value.

In more recent times men like J. Clerk Maxwell, Castigliano and Mohr have chiefly been recognised for their work in the analytical field and one does not associate them with any large or famous constructions. In fact, most of the large structures have been the responsibility of men who have preferred to adopt a retiring attitude and who were not exceptional mathematicians.

Are these men noted for their brilliant contributions in the academic field [4]? On the contrary, they would most probably consider themselves very weak academically.

*Presented at the Midland Counties Branch of the Institution of Structural Engineers on 26th March, 1954.

Today, as ever, the academic engineers produce their researches and other contributions to knowledge based largely on theory and, to a lesser extent, on experiment whilst the practising engineers still seem to lag behind. The lag, the existence of which is doubtful, is not as great as it used to be owing to the better liaison which now exists between the academic and the practical worlds.

It is, of course, understandable that practice should often precede theory. It is quite logical to build some new form of structure and to test it before attempting to use it rather than to work to a theory which often has its limitations. In this way, we often find ourselves inventing a theory to fit our practical experience. More often than not this theory is very complicated unless we can make simplifying assumptions. Even after considerable work our theory may be found to be incomplete and, perhaps, difficult to finalise.

An example of this may be found in the work on struts. After Euler, both Perry and Robertson made their respective contributions and we have for many years used the Perry-Robertson formula in our British Standards and Codes of Practice. Present methods of designing struts are still often inadequate and unrealistic, however, as are our methods of designing building frames.

By and large, structural engineering is still an art to be acquired by practical experience and not by reading books or learning theories—important as they are. We still rely so much on our materials and workmen that we would do well sometimes to reflect on the difficulties of our forebears who had to work with cast or malleable iron on whose strength or soundness they could not always rely.

The importance of having a stiff structure as well as a strong one was realised a century ago. The academic engineers gave attention to this aspect of structural design in quite early times. Hooke (1635-1703) was one of the earliest to consider strain although others had done so before him. The subject is, however, one which on the practical side has been neglected long enough and further thought is urgently required.

The L.C.C. regulations of 1909 permitted a beam to deflect one-four-hundredth of its span. Today we allow one-three-hundredth of the span. This seems to be practically the sum total of our progress on the stiffness of structures.

On the theoretical side, however, the same is not true since there are available adequate methods for calculating the strains and deflections in both determinate and indeterminate structures provided the physical data is available.

The Practical and Experimental Approach

Within the last decade there have been some enormous developments in both the practical and theoretical fields. On the practical side there have been new methods of fabrication introduced of which the chief are electric arc welding and gas cutting. The introduction of welding has not only increased the efficiency of steelwork but has very much enhanced its scope as a structural material. There were, in the early days, very many objections to the introduction of welding, but in spite of this the process has had a wider and wider use until today it is recognised as one of the standard methods of fabrication. The introduction of welding has also enabled rigid framed structures to be employed and whilst these are more difficult to desi they enable economies to be made in the weight of st required.

An extremely important development since t second world war has been the production of mild st of British Standard 15 quality with a guaranteed yi point. In this connection it is interesting to note th during the last twenty-five years of progress t emphasis in design has been steadily removed from t ultimate strength of mild steel and focused on t yield point since the latter more accurately represen the design criteria controlling load bearing capacity.

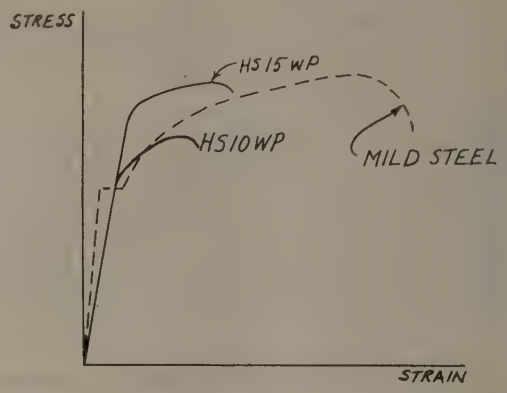

Fig. 2

Mention must also be made of the use of aluminiu alloys. Fig. 2 shows typical stress/strain curves f aluminium alloys and compares them with that f steel. The differences in characteristics of these tw types of material will be observed. Whilst mild ste has a well defined yield point and plastic zon aluminium alloys are characterised by a more gradu increase in rate of strain at the so-called yield. Furthe more, although the ultimate strengths of aluminiu alloys and steel may not be greatly different there a large differences in ductility as measured by elongatio over original gauge length.

These properties are important to the designer wh especially in choosing a suitable aluminium alloy for structure, has to distingish between the relative adva tages of high strength or ductility since the high strength alloys tend to have lower ductility.

Although, at the present time, there has not been ar serious suggestion in regard to the application of t plastic theory to aluminium alloy structures, one coul justifiably, expect attention to be given to this matt if the employment of such alloys were to increase co siderably.

More Recent Research Work

It is understandable that research work both Britain and abroad on the load bearing capacity metal framed structures should have been directed the first instance towards the strength of connection Reference to the " First Report of the Steel Structur Research Committee "[3] will show that in 1931 t following items were selected as being most deservi of attention from the research point of view.

(a) Loads on Floors ;
(b) Strains in Buildings ;
(c) Stress Analysis ;
(d) Specification and Properties of the Material ;
(e) Welded Connections ;
(f) Vibrations.

In the same report the late Sir Reginald Stradling pointed out that the research would take considerable time and the only items on which immediate reports could be made were " Loads on Floors " and " Working Stresses." At the end of the First Report there was included a proposed Code of Practice in which lower floor loadings and higher working stresses in steel were two important items. On reading this report one gains the opinion immediately that the Steel Structures Research Committee had no illusions regarding the difficulty of translating the results of their research work into a form suitable for use in the average design office.

It is indeed unfortunate that the design and the experimental approaches have grown up somewhat independently of each other with the result that both have become very fixed in their ideas regarding structural steelwork design. The rank and file of designers (chiefly to be found with structural steel fabricating firms) are loathe to depart from the traditional methods of design which assume pin-jointed connections. On the other hand the student of the Steel Structures Research Committee Reports whilst being happy in the knowledge that he could design structures with semi-rigid connections would shudder if asked to be put into serious competition with his statically determinate contemporaries.

Although a great deal of attention was focused on the development of a method of designing building frames with semi-rigid and fully rigid connections, the Steel Structures Research Committee also devoted a considerable amount of attention to the strength of beam to stanchion connections. The late Professor Batho spent a considerable amount of time in examining riveted and bolted stanchion connections and a great deal of his work is written up in the Second Report of the Steel Structures Research Committee. Batho was able to devise a method of designing building frames which he called the " beam-line method " which, even today, appears to the Author to be the best method of designing frames with semi-rigid connections.

In the early nineteen thirties we also find the beginning of serious attention in England to a development which is now called the Plastic Theory. Work had already been carried out, however, by Kazinczy, Bleich, Emperger, Fritsche, Maier-Leibnitz and others, details of whose work were published in the Second Report of the International Association for Bridge and Structural Engineering. Following this, J. F. Baker and others have investigated the ultimate load or plastic method and carried out extensive experiments on full scale frames.

There is no doubt that nearly all research work can help the engineer to obtain a better understanding of the behaviour of structures under load (dare one say no matter how badly carried out), and it is interesting to note the results of tests published by E. S. Andrews in the Second Report of the I.A.B.S.E. and to use a combination of the beam line and plastic theory and interpret his test results. (Appendix I.)

More recent work on Aluminium Alloy Structures at the University of Birmingham has indicated that for cleated beam to stanchion connections in aluminium alloy the principles of the beam line method may be applied.[6 7] In this research considerable use has been made of prototypes and the Author is of the opinion that the experimental examination of such prototypes is virtually important in such work. It is, moreover, of interest to note that a great deal of the work undertaken by the Steel Structures Research Committee and by the British Welding Research Association has been on actual frameworks or models of structures and that all theoretical conclusions are, as far as possible, backed up by experimental results.

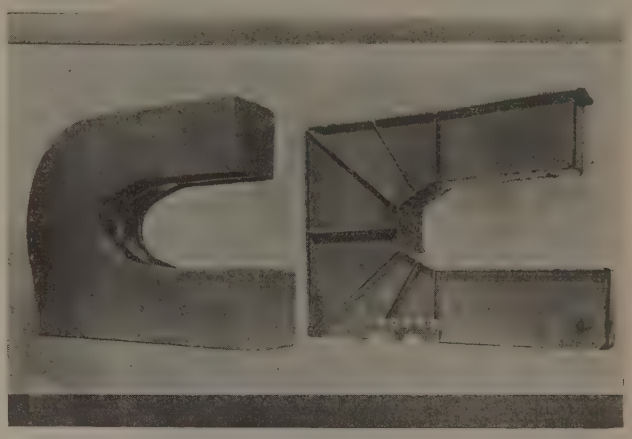

Fig. 3

The use of models need not only be of value in fundamental research but may also be of great use in solving day by day problems. An example of this is shown in Fig. 3 which indicates two comparative 1/8 F.S. models of 250 ton press frames which the Author tested some years ago. The results were extremely valuable and enabled considerable economies to be made to the final design of the structure. The two designs were for three and two-web structures, respectively, the former having a rounded profile and the latter an angular profile. The latter involved considerably less weight of steel than the former and lower fabrication costs in spite of the fact that it exhibited a much greater carrying capacity and stiffness.

Theoretical and Analytical

Long before the formation of the Steel Structures Research Committee it had been realised that there were certain reserves of strength in structures which were not taken into account in ordinary design calculations. These reserves of strength have become more and more evident as we have reduced our factors of safety and worked closer to the collapse load condition.

The effects of aerial bombardment amply demonstrated how steel structures could remain in position to perform their load-carrying function after removal of members which reduced them to *less than* " just-stiff " frameworks.

Fig. 4 shows part of a 10-storey cased steel frame block of flats in which the foundations of the corner wing were damaged by a bomb in 1941. The building was knocked approximately 1 ft. out of plumb. The severed corner stanchion was temporarily propped and subsequently supported with a brick pier. It remained in this condition for about 12 months prior to the whole wing being pulled down and rebuilt.

The 8-storey cased steel frame building, part of which is seen in Fig. 5, was damaged by a bomb in 1941. The end support of an encased 5 ft. deep steel plate girder spanning the full width of one end wing was

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Fig. 4

blown away. The girder collapsed but the building remained plumb and there was no other structural damage. The end of the girder was temporarily shored and remained in this condition for some months prior to the damaged portion being demolished.

R. Levi pointed out in general terms in 1935 tha even statically determinate structures have enormou reserves of strength whilst statically indeterminat structures have even more. J. F. Baker and his Cam bridge research team have since shown the latter to b true for a number of specific examples and hav developed a method of predicting the collapse loads o rigid framed structures. In this connection it has bee shown that the reserves of strength tend to increas with increase in redundancy. Furthermore, collaps does not occur when the structure or one of its com ponent parts first reaches the yield point but there a redistribution of stress to less highly stressed parts o the structure. This redistribution process continue right up to collapse of the structure. But it must b remembered that when a highly redundant structur reaches the collapse load considerable deformation may have taken place and the structure may hav taken on an alarmingly distorted appearance.

The plastic theory of design takes account of thes reserves of strength and enables the engineer to mak close estimates of the ultimate load bearing capacity o structures. This theory is known by several descrip tions, e.g. limit design, ultimate load method, collaps load method.

We must, therefore, bear in mind that when w calculate collapse loads we must also visualise an calculate the various stages up to this condition. W must also ask ourselves very seriously whether or no we are prepared to accept the fact that, during th working life of the structure, part of it will reach th yield stress and suffer permanent deformation. If no then we are still only prepared to design within th elastic range and, although we may calculate collaps loads as an additional criterion of design, we are stil adhering, fundamentally, to elastic design.

If, on the other hand, we are ready to accept yieldin in certain parts of the structure (say, in beams only we must decide how extensive and with what effects o other parts of the structure we will allow such yielding

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Fig. 5

The Author sees considerable dangers in the universal adoption of the ultimate load theory as a method of design. It is comparatively easy to predict the collapse load on simple structures for given loading conditions but it is not always so easy for the designer to check through the various stages of " hinge " formation leading up to failure.

Appendix II gives typical examples in the latter connection.

It is strongly recommended, therefore, that structural engineers employ the " plastic " method only with full realisation of all it implies and recognise that the elastic theory is not superseded by the " plastic " theory but becomes more essential than ever if a full understanding of load bearing capacity is to be obtained.

Whilst we have learned a great deal by patient experiment and research we still seem only to be able to take advantage of our knowledge by simple expedients. Most of our experience is devoted to the improvement of specifications for structures designated as " simply designed " and our Codes of Practice still say very little about semi-rigid and fully rigid constructions.

Why this state of affairs exists is not clear since there must be many occasions when advantage could be taken of designing both rigid and semi-rigid constructions if up-to-date and suitable codes of design practice were to hand.

The lessons from past experience indicate that first principles are always of supreme importance in both design and analysis. To understand the true behaviour of structures it is necessary to experiment with prototypes of models, whilst to improve existing designs it is necessary to expend reflective thought and effort—not always induced by competitive design.

There would appear to be no reason why we should not be clear-cut in our ideas as to when we should aim at extreme accuracy in design or when ordinary design methods are good enough ; as to when the " plastic " method ought to be used and to what extent it is advisable to recommend it for general adoption in industry ; as to when should the semi-rigid type of construction known as structural steelwork be treated as such and when should we put on "blinkers" and design it as simply supported.

All these questions could be thrashed out not only for the benefit of those who take an academic interest in structural engineering but also for those who make a livelihood.

Reliability of Steelwork

Future Trends

It has been stated that the future of structural steel in building may be in jeopardy because of prestressed concrete, but one doubts this very much indeed. Steelwork must remain for many years to come by far the most reliable of structural media. But designers must not be content to remain static in their ideas. Certain items cry out for urgent research such as tubular welded connections and the design and construction of welded truss bridge connections. Moreover, it still remains urgent to consider deflections in structures and their components although work on this would have fairly general application to all types of construction.

Deflections

The Author feels that before very long we shall find ourselves using load factor methods of design as a matter of course. It seems the only logical way of embracing ultimate load theory into our design methods.

As we have increased the slenderness of our constructions new problems have arisen and we now have to consider the danger of instability as well as excessive deflections. This applies particularly to structures in light alloys where Young's modulus is one third that of steel and also to structures in thin material, such as cold rolled steel. It would seem, therefore, that, if we are to make the best use of such new materials we must face up to the introduction of some more mathematics into day to day practice.

There appears, to the Author, to be a much better chance of developing good practice with new materials of construction than with the older ones where traditional methods die hard. This applies particularly to aluminium alloys and lightweight cold rolled steel constructions. Here there is ample evidence that structural engineers are making a serious effort to strike the correct balance between theory and practice in regard to both of these materials.

One would have expected that already high tensile steel would have had a wider use and that such steels would have shown considerable economies for certain types of structures. The exact reason for this is not clear and it is hard to believe that the cost of the steel itself renders it uneconomical. Experience with bridges showed that high tensile steel has no advantage where deflections are critical and construction depths limited, but other types of structure and bridges also have demonstrated its value where stress is the limiting criterion. Perhaps high tensile steel will come into its own with the prestressing of steel or aluminium structures.

It is interesting to conjecture whether other metals may not be used for structures in time to come. There is no doubt that aluminium alloys would become plentiful and reasonable in cost if atomic energy gives us cheap electric power. Titanium has also been suggested as an ideal structural material, but one cannot see it being used extensively within the next generation.

One thing in the future seems to be sure and this is that, in our time, there will be little or no relaxation of effort to produce more economical metal framed structures designed to more scientific and to closer limits.

Appendix No. 1

(Notes on tests by E. S. Andrews, Esq., published in the Second Report of the International Association for Bridge & Structural Engineering, Berlin-Munich Congress, " Preliminary Publication," 1936.)

The form of test specimen was as indicated in Fig. 6 and consisted of 4 in. × $1\frac{3}{4}$ in. × 5 lb. I sections, the properties being as follows :

	X—X axis	Y—Y axis
Second Moment of Area	3.66 in.^{4}	.186 in.^{4}
Section Modulus	1.83 in.^{3}	.213 in.^{3}
Area	1.47 in.^{2}	

The beams were connected to the stanchions by top and bottom cleats 2 in. × 2 in. × $\frac{1}{4}$ in. Mild steel bolts $\frac{5}{16}$ in. in diameter were employed.

Two frames were tested and a single beam of the same section. The results of the tests are reproduced herewith (Figs. 6-9 incl. by permission of the International Association for Bridge and Structural Engineering.)

If the maximum single beam load is taken at 3.75

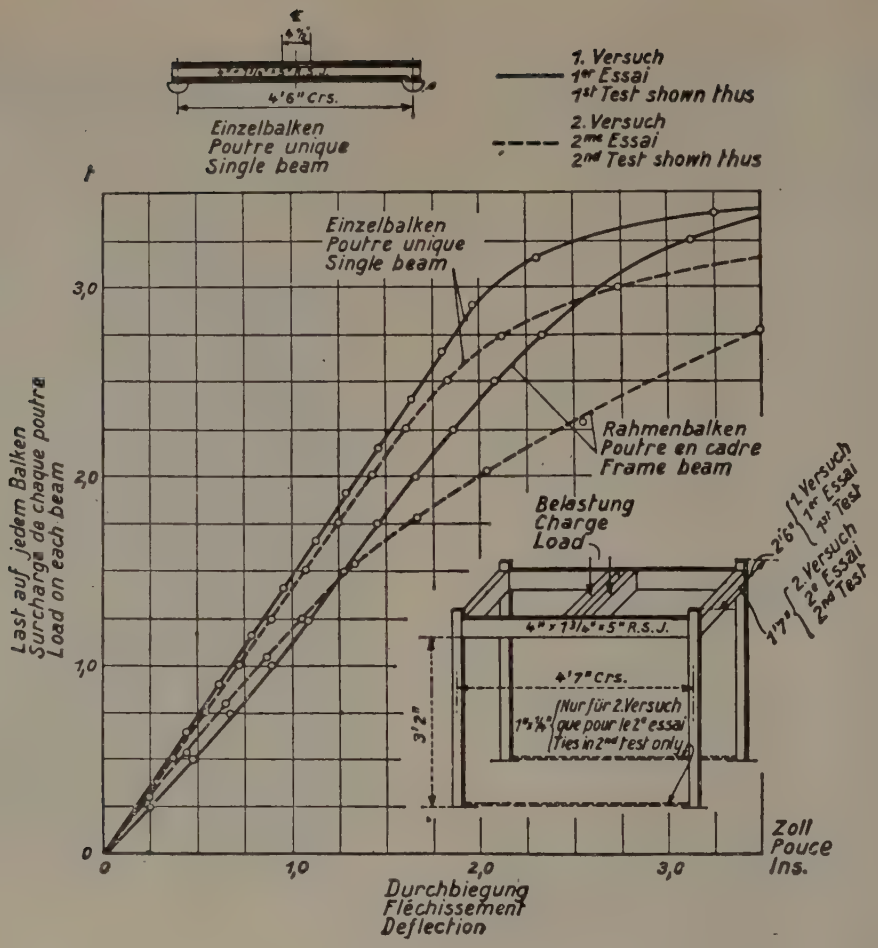


Fig. 6

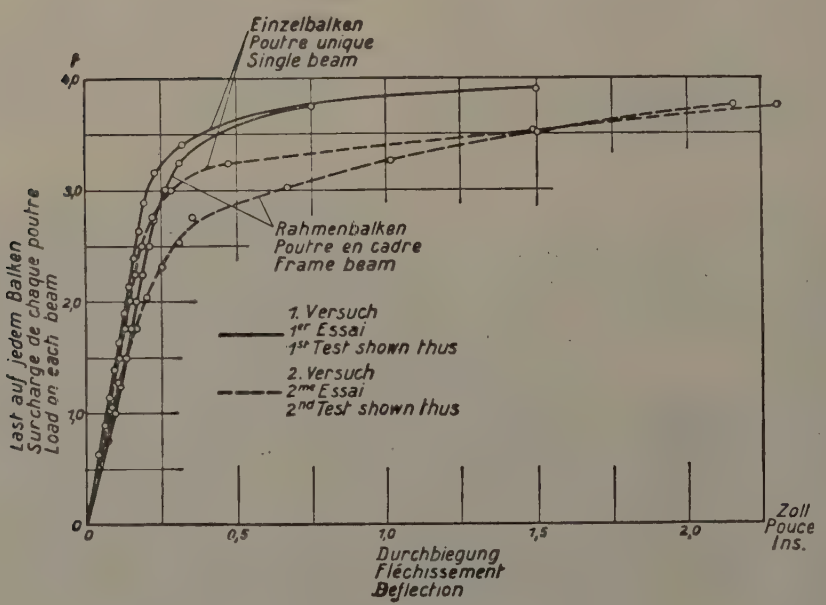


Fig. 7

tons (the result of the second test) the full plastic moment on the section at failure is equal to

$$M_p = 3.75^T \times 4.5 \times \frac{12}{4}$$

$$= 50.6 \text{ Ton. in.}$$

Approx. Yield point of steel derived

$$\text{from } M_p = \frac{50.6}{1.83 \times 1.15} = 24.0 \text{ T/sq. in.}$$

Approx. Yield point of steel derived

$$\text{from } M_y = \frac{2.9^T \times 4.5}{1.83} \times \frac{12}{4} = 21.4 \text{ T/sq. i}$$

E. S. Andrews stated in his paper that the fram were designed in accordance with the approxima methods of design in use at the time which, in fact, we very similar to our present day B.S. 449 method. found that the frames did not behave as predicted such design calculations or as predicted by assum rigid jointed portal frame structures.

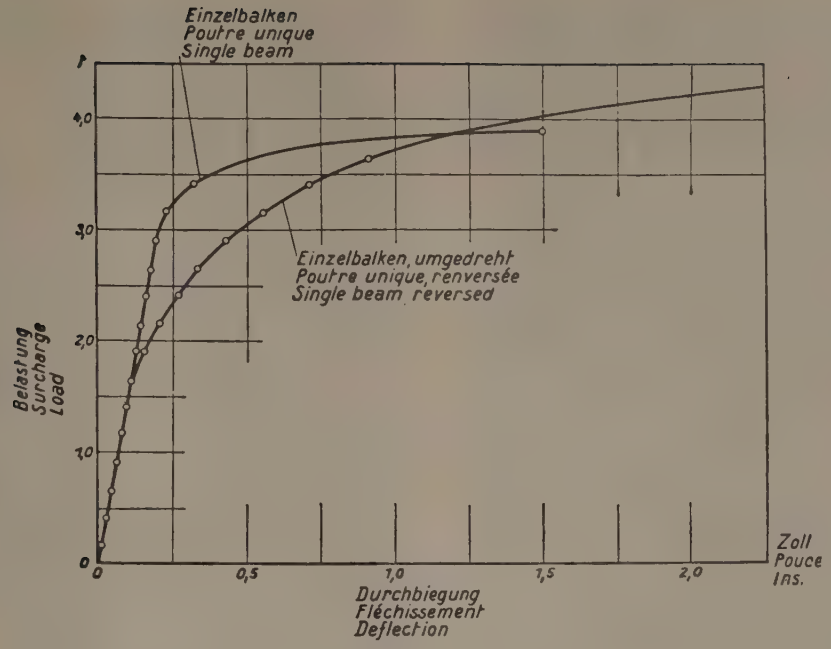


Fig. 8

By employing a combination of the beam-line method and ultimate load theory it is possible to substantiate his test results in the light of present-day knowledge.

Andrews showed that there was very little difference between the load bearing capacity of the framed beam and that of the simple beam. The results of his first test with frames were stated to be somewhat unreliable but his Test 2 results were presumed to be accurate.

Fig. 9

An examination of the frame in the light of the ultimate load method will indicate that, for collapse, plastic hinges must form at the centre of the beam and either the ends of the beam or the heads of the stanchions. Reference to Fig. 9 will indicate that this did, in fact, happen and that the cleated connections were sufficiently strong to transmit the collapse load moments of the stanchions about the Y—Y axis. The latter value is equal to

$$M_p = fy \times 0.213 \times 1.50$$

If fy is taken at 21.4 tons per sq. in.

$$M_p = 6.84 \text{ tons in.}$$

It should be noted that, had the cleats not been strong enough to transmit the collapse moment to the stanchions, there would, in effect, have been plastic hinges set up in the cleats. Under these conditions the collapse would have been by hinges forming in the cleats and at the centres of the beams. No failure of the stanchion would have taken place unless the cleats had been able to transmit the collapse moment.

The frames were able to support an ultimate load of 3.95 tons as compared with a simple beam load of 3.75 tons. The difference in carrying capacity was made up with the ultimate bending moments of resistance of the stanchions.

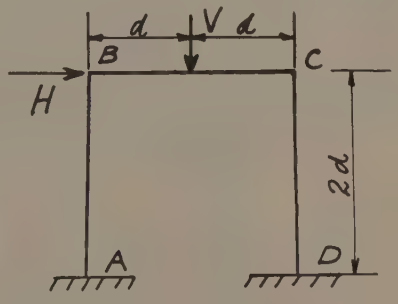


Fig. 10

Appendix No. II

Analysis of portal frames under load.

Determination of loads at which successive hinges are formed.

If a fixed based portal frame as shown in Fig. 10 is considered it is proposed to calculate the loads at which plastic hinges will form when various ratios of $\frac{H}{V}$ are chosen.

It has already been shown by Professor J. F. Baker that the appropriate modes of failure for such a frame would be as in Table 1.

By employing the values indicated in Table 1 it is possible to determine the mode of collapse for any square portal with constant second moment of area

provided it is subjected to a vertical load at the centre of the beam and a horizontal load along the beam.

It should be borne in mind, however, that different loading systems would almost certainly produce different modes of failure and necessitate careful investigation to determine the positions of the plastic hinges. Likewise, variable second moments of area would also entail a special approach.

In practice, the structural engineer is seldom faced with designing rectangular portal frames with loadings as indicated above and cannot, therefore, employ such ready made answers. It is, in fact, necessary to investigate any unusual frame from first principles.

The following calculations show how the plastic hinge formation might have been investigated for two typical cases mentioned above (Modes 2 and 3—Table 1). The principles may be extended to other examples which it is not possible to mention here.

The calculations, as set out below, are for a portal frame with constant section and the height equal in magnitude to the span. They have been verified experimentally with models of $\frac{1}{2}$ in. $\times$ $\frac{1}{4}$ in. mild steel with a span of 10 inches and an effective column height of 10 inches.

Professor Baker has given details (see Table 1) of the various modes of collapse for varying proportions of horizontal load applied along the beam to central vertical load. These modes apply at collapse but it is proposed to discuss one or two proportions of loading and to determine what happens as the loads are gradually increased from zero up to collapse.

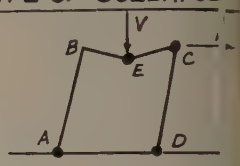
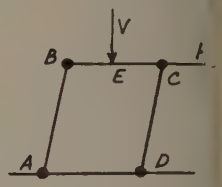
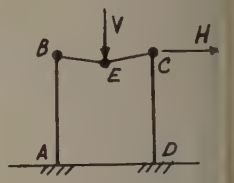
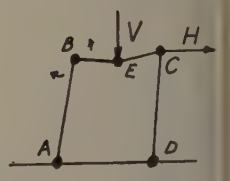
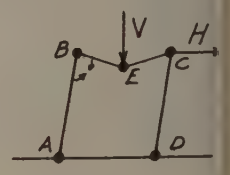
MODE	CRITICAL VALUES	TYPE OF COLLAPSE
1	$V > H > \frac{V}{4}$	
2	$H > V$	
3	$\frac{V}{4} > H$	
4	$H = V$	
5	$H = \frac{V}{4}$	

Table 1

Case 1. *Central point load only.*

The frame is loaded as shown in Fig. 11(a) with a central vertical load (V) and with no horizontal load along the beam and the value of V is gradually increased. The mode of collapse will be No. 3, Table 1.

As long as the stresses in the frame are wholly within the elastic range the bending moments will be in proportion to those shown in Fig. 11(a). The moments around the frame will increase proportionately until the

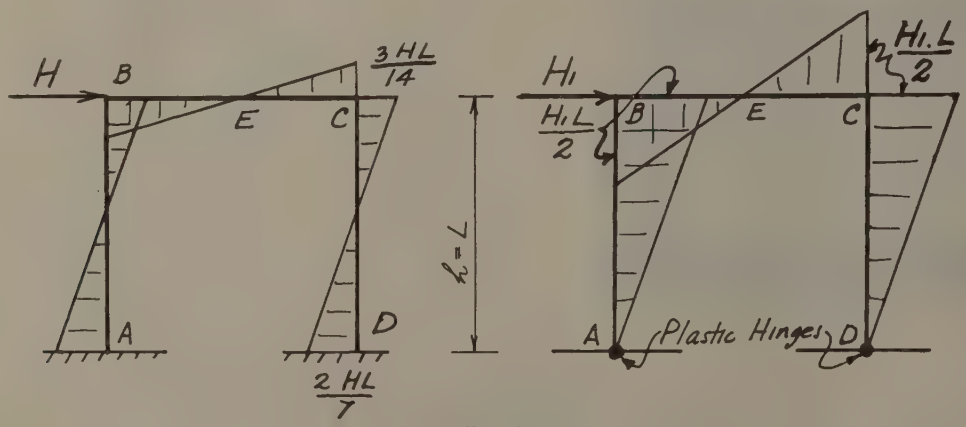


Fig. 11

cross-section at E has yielded fully and become, in effect, a plastic hinge. This occurs when the moment at E is equal to 425 lb. in., the fully plastic moment of the section (see Fig. 12).

When this condition is reached, the section at E will not be able to carry any more moment and any *further* resistance to bending must be provided elsewhere. In respect to *further* loading the structure will now behave as if it were hinged at E ; the distribution of bending moments will now be as shown in Fig. 11(b) for any loading *above* that required to produce a hinge at E.

The loading on the structure may now be increased until the sum of the residual moments at B and C the moments $\left(\frac{V1 \cdot L}{4}\right)$ created by the additional $V1$ are together equal to the full plastic moment of section.

When the hinge at E occurs, the bending momen E is equal to $\frac{VL}{6}$

$$\frac{VL}{6} = 425 \text{ lb. in.}$$

$$\text{or } V = 255 \text{ lb.}$$

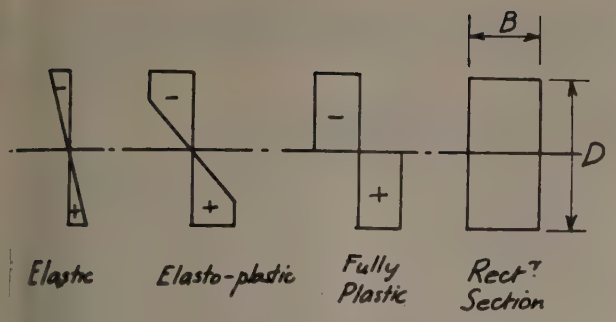


Fig. 12

t this stage,

$$M_B = M_C = 212.5 \text{ lb. in.}$$

o produce collapse $M_B = M_C = 425 - 212.5 = 212.5$ lb.

$$\frac{V1L}{4} = 212.5 \text{ lb. in. or } V1 = 85 \text{ lb.}$$

The total load required for collapse is, therefore,

$$255 + 85 = 340 \text{ lb.}$$

As a check : twice the plastic resistance moment of he section should equal $\frac{WL}{4}$, the free moment on the eam.

$$\frac{340 \times 10}{4} = 850 \text{ in. lb.} = 2\,M_p.$$

ase 2. Horizontal load only.

The frame is loaded as in Fig. 13(a) and the value of *H* gradually increased. Plastic hinges will form, first of all, at *A* and *D*.

When this occurs,

$$\frac{2}{7} \times H.L. = 425 \text{ in. lb.}$$

$$H = 149 \text{ lb.}$$

The residual moments at *B* and *C* are equal to

$$\frac{3}{14} H.L. = 319 \text{ in. lb.}$$

The final stage of collapse is when plastic hinges are at *B* and *C*.

In the second stage of loading the frame behaves like a two-pinned portal.

$$\frac{H1 \,.\, L}{2} = 425 - 319$$

$$H1 = 21 \text{ lb.}$$

$$H1 + H = 170 \text{ lb.}$$

General Observations.

The above two examples are comparatively simple but other forms of loading would produce more complex conditions of hinge propagation making analysis of these intermediate conditions extremely tedious and, sometimes, very difficult.

It is hoped that sufficient explanation has been given to demonstrate that in order to investigate the hinge propagation in detail, it is necessary to employ elastic analysis very extensively.

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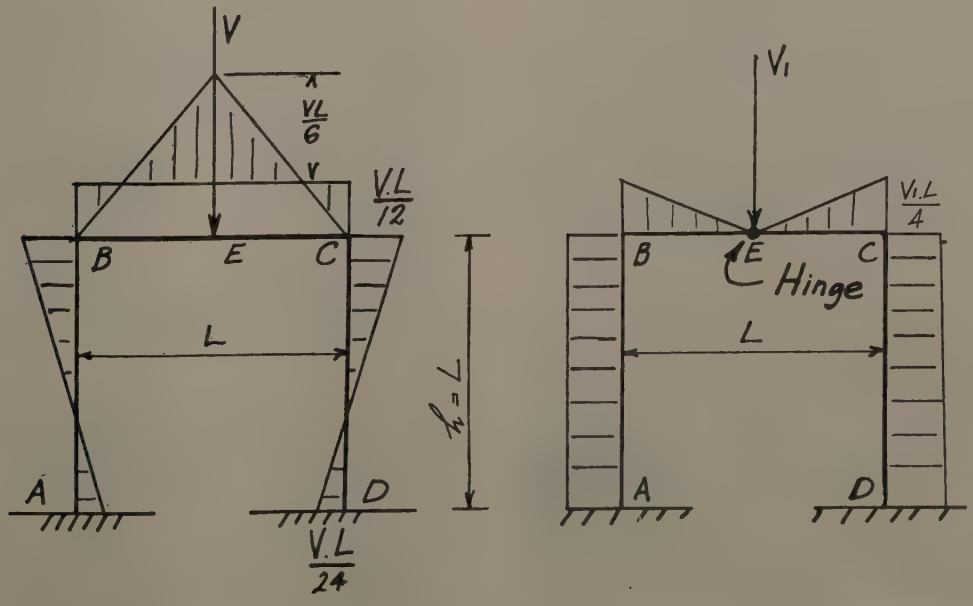


Fig. 13

The Process Building at Capenhurst: Some Structural Problems*

By T. C. Waters, M.I.Struct.E.

Synopsis

THE buildings for the Diffusion Plant at the atomic energy factory at Capenhurst provided the structural engineer with some interesting problems.

The speed at which the factory had to be built in no way reduced its interest to the designer who had to keep his mind fluid and his designs flexible to ensure wherever possible that modifications or additions could be incorporated during construction, without prejudice to the unalterable date—the start up of production.

During the course of the job considerable use was made of cold formed strip steel and also of tubular construction, in an effort to save steel.

The paper attempts to present some of the answers to the problems that had to be considered in the design of the Process Building and the Cells inside it, and refers to some of the structures within the services buildings associated with it.

The Gaseous Diffusion Process

Before proceeding, it would be as well to dwell for a few moments upon the principles underlying the gaseous diffusion process which dictated the size of the building and the nature of the problems presented to the structural engineer.

When a gas containing two different molecules diffuses through tiny holes in a porous membrane the lighter molecules come through faster than the heavier ones, consequently the gas on the far side is enriched in the light molecules. If this diffused gas is pumped away after a short time, the fraction removed will be richer in the lighter molecules than the fraction left behind. The gas remaining, the heavier fraction, is only slightly depleted and still contains a very great number of light molecules. It is therefore recycled, that is to say it is pumped to another " stage " preceding the one where the process began and later allowed to diffuse through another membrane. The resulting diffusate now enriched in the light molecules is thus pumped to the input of the membrane at which the whole process started. The principle is shown diagrammatically in Fig. I.

The molecules of uranium 235 and 238 differ only by about 1 per cent., so the effect of this diffusion process upon these through one membrane is very small, but when the diffusion is repeated by pumping the slightly enriched gas away and passing it through a second porous membrane, and then a third, and then a fourth, and so on many times, the enrichment can be increased to any required degree of purity. It will be clear, of course, that as the gas travels up the pl through membrane after membrane its volume decrease, since the bulk of the original gas was m up of heavy molecules. The process aims at prod ing uranium 235.

The gaseous compound used at Capenhurst is ur ium hexafluoride (known as Hex.), containing urani 235 and 238 in proportions of approximately 1 to 1 which is gaseous at temperatures around 140°F a solidifies at 100°F. It is highly corrosive and rea very readily with a great many substances includ water, to form solid compounds. The whole pl must therefore be perfectly clean and free from a leak through which water vapour or any other c taminant might enter.

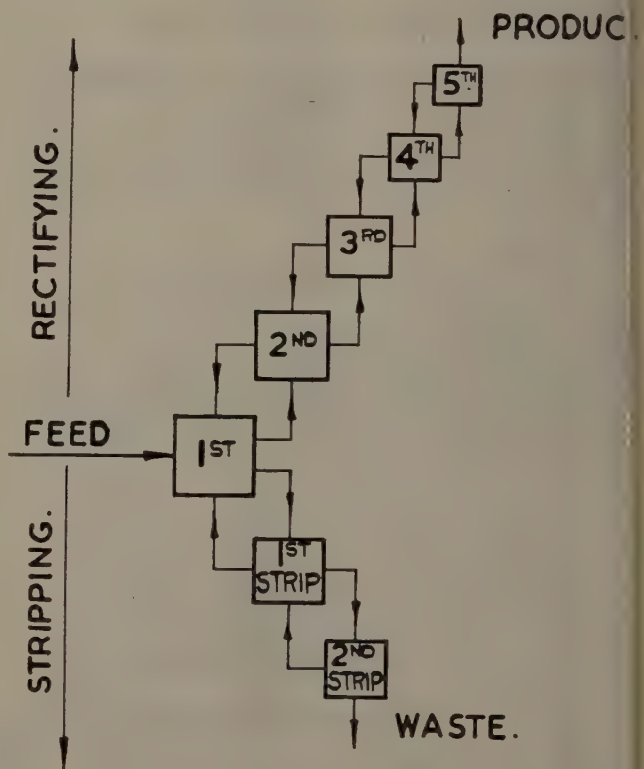


Fig. 1.—The Diffusion Principle

To achieve suitable working conditions therefo the plant is made vacuum tight and in addition, has to be kept at a tropical temperature to ensure th the compound does not condense.

The diffusion plant is accordingly based on a sim physical principle and the whole process is carried c in one continuous operation by very large numbers units or " stages " all performing the same functi

* *Paper read before a Joint Meeting of the Lancashire and Cheshire Branch of the Institution of Structural Engineers and the North Western Association of the Institution of Civil Engineers on the 14th February,* 1955.

ad the fact that the process gas goes from one stage t the next throughout the plant means that it must b constructed under one roof and for ease of main- tiance and operation at one floor level.

The Process Building

The process building which houses the plant is a l:ge single storey steel framed shed with 12 ton over- lad electric cranes for handling electrical and mech- ical equipment. Inside, the diffusion plant is loused in rows of insulated boxes or " cells." Inside le cells the " stages " are arranged in groups varying i size according to the position they occupy in the ffusion process.

Periodically, maintenance is necessary to the 'stages." This involves isolating the " stage " from ie process, breaking the seal, removing the " stage " the workshops, inserting a fresh " stage," sealing id bringing it into the process line in the shortest ossible time. This involves a high degree of inter- hangeability of plant and of accuracy in floor slabs hich carry the " stages."

In spite of the high operating temperature of the as, large quantities of water are used to cool it after ompression. This is brought into the system in over- ead pipes slung from the building frame and returned elow ground to Pump Houses and thence to Cooling owers where it is cooled and recirculated through the lant.

The process building with its workshops and asso- iated services buildings was erected in 3 phases about qual in size, and together they now cover an area of ver 25 acres. Phase 4 has yet to be constructed.

PHASE 1

Conditions laid down.

These were as follows :

1. Centres of main stanchions over Cell area to be 117 ft. 0 in., the truss main tie and corridor roof beam to be 44 ft. 6 in. above finished floor level.
2. Width of corridors for access, maintenance, and plant handling purposes to be 19 ft. 6 in. centres of steel, and gridding of steel along the corridors to be based on 22 ft. 6 in.
3. Rainwater gutters to be kept away from cell areas. To meet this requirement it was decided to cantilever the truss rafters over the top of the corridors to ensure a quick run off of storm water.
4. As much free space to be provided in the roof area above the main tie level for ventilation trunking should it be needed. This condition was met by using a 3-pinned tied arch or truss at 22 ft. 6 in. centres, with intermediate rafters. The main tie was lowered in order to increase this " free " area, and to lessen the boom loads in the truss system.
5. Provision to be made for slinging cables, walkways, pipe services, etc., from the main tie, with connections to plant and equipment at cell top level. It was expected that these loads would eventually be equivalent to about 8 lbs. per sq. ft. of plan area.
6. Because of the pipe services between truss and equipment on the cells, and because of the importance of achieving vacuum conditions, the plant designer was eager for assurances from the structural engineer that there would be no differential movement as between truss and cell top, but after some hard bargaining it was eventually agreed that lateral and vertical movement of the roof from a combination of all causes should not exceed $1\frac{1}{2}$ in. The problem then was :

   (*a*) To design the roof and then to calculate the vertical and lateral deformation under load, and

   (*b*) To design the corridor portals and then to assess the horizontal deformation under the worst conditions of load and temperature.

Design Generally

Temperature.

Apart from climatic changes, the question of internal temperature conditions was unknown in the early days of design. The problem of heat outside of the cells but inside the building proper, and the need or otherwise for special ventilation was as yet unsolved, and it was not known to what extent the running temperature of the plant would influence the general temperature conditions within the building.

Expansion Joints.

In spite of the foregoing doubts, it was clear that the temperature question was a real one from a structural view point, and it was therefore decided to provide expansion joints across the building and along the building, so that for design purposes and in particular for stability against wind and temperature, the building was broken down into units approximately 410 ft. 0 in. × 180 ft. 0 in. embracing three roof spans and three corridors in width, and 8/22 ft. 6 in. bays longitudinally, as shown in Fig. 2.

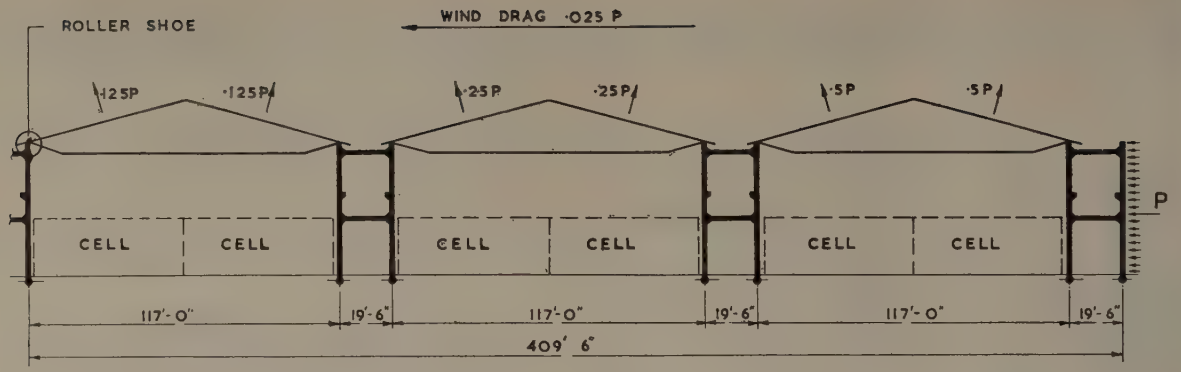
The Bracing System and General Stability.

For transverse stability the corridor steel bent was designed as a two storey portal, three of which, in combination laterally, were designed to resist forces due to temperature movements, wind and drag etc. Elastic deformation of the roof trusses under vertical loads, wind, and temperature changes, influenced the deformation of the portals, and were therefore important features in the design of the latter. For longitudinal stability, vertical bracings were provided centrally in the unit along the lines of the corridor to transmit wind loads to the foundations. Horizontal counter bracings at eaves level were provided across the corridor in the wind bay to act in conjunction with the vertical bracings.

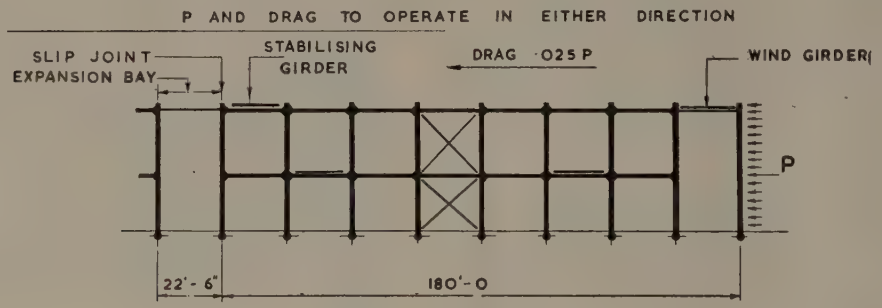
Gable bracings were located as close to the centre of the unit as was possible, to ensure that thermal stresses in the bracings were kept to a minimum.

The expansion joints in the length of the roof raised the question of instability in the trusses adjacent to them and between them. This was dealt with by the provision of stabilising girders at main tie level designed to carry forces induced by wind drag, such forces being transmitted to the foundations through the eaves channels and the vertical bracings.

Rafter bracings were designed and proportioned effectively to stabilise the roof construction in general and the rafters in particular.



TYPICAL CROSS SECTION OF UNIT SHOWING WIND CONDITIONS.



TYPICAL LONGITUDINAL SECTION OF UNIT ALONG CORRIDOR.

P AND DRAG TO OPERATE IN EITHER DIRECTION

Fig. 2.—The Design Unit

Structural Design

Generally in accordance with BSS.449(1948).

Wind Velocity — 75 m.p.h.

p = 18.5 lbs. per sq. ft. on the side of the building.

p = 19.6 lbs. per sq. ft. on the gable.

Suction = 0.5p = 9.25 lbs. per sq. ft. on the windward truss.

Drag = 0.025p = 0.46 lbs. per sq. ft.

Young's Modulus for steel taken as 13,400 tons per sq. in.

Temperature—A range of 60°F.

Coefficient of expansion for steel 0.000007 per °F.

Truss Design.

In view of the fact that the truss was to be carried upon flexible supports which in their turn were influenced by the elastic movement of the truss, it was important to determine not only the loads in the various truss members for design purposes, but having designed them, to determine the maximum vertical and horizontal deformation of the truss under the various combined loadings.

The following combinations of loads were considered in order to arrive at the worst design conditions which naturally included those tendencies towards reversals of stress in tension members.

i. Dead + Snow + Full Services.

ii. Dead + Snow + Full Services + Suction wind and drag.

iii. Dead + Half Services + Thermal + Suction wind and drag.

iv. Dead + Half Services + Suction + wind a drag.

v. Dead + Thermal + Suction + wind and dra

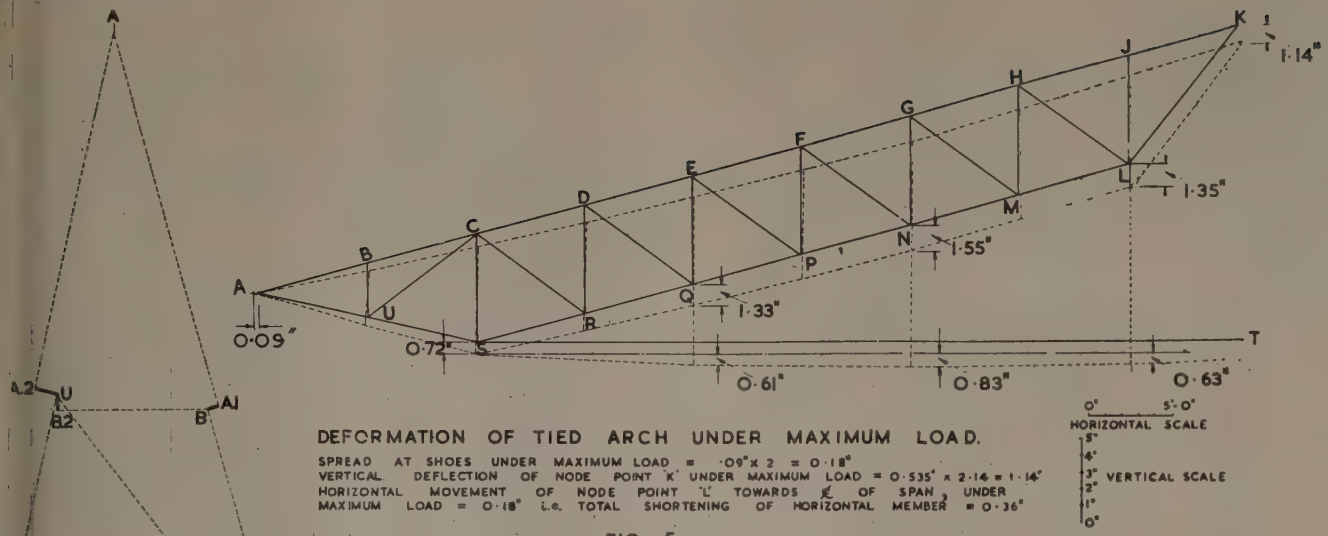
vi. Dead + Snow + Services — Thermal.

The roof cladding chosen was R.P.M. V Beam shee ing, as its deep flute offered the best chance of throwi off rainwater from the rather shallow pitch of 15°, a was also capable of spanning between purlins spac at 6 ft. 9 in. centres. Purlin positions were made c incident with truss node points to eliminate seconda bending in the rafters.

The following plan loadings were used for design :

Dead loads	18	lbs. per sq.
Services .....up to	8	lbs. per sq.
Snow	12½	lbs. per sq.
GROSS TOTAL	38½	lbs. per sq.

The truss is statically determinate and was design in the normal manner. Force diagrams for De loads, Suction, and Unit Thrust were drawn. Forc



DEFORMATION OF TIED ARCH UNDER MAXIMUM LOAD.

SPREAD AT SHOES UNDER MAXIMUM LOAD = ·09" x 2 = 0·18"
VERTICAL DEFLECTION OF NODE POINT 'K' UNDER MAXIMUM LOAD = 0·535" x 2·14 = 1·14"
HORIZONTAL MOVEMENT OF NODE POINT 'L' TOWARDS ℄ OF SPAN, UNDER MAXIMUM LOAD = 0·18" i.e. TOTAL SHORTENING OF HORIZONTAL MEMBER = 0·36"

FIG. 5.

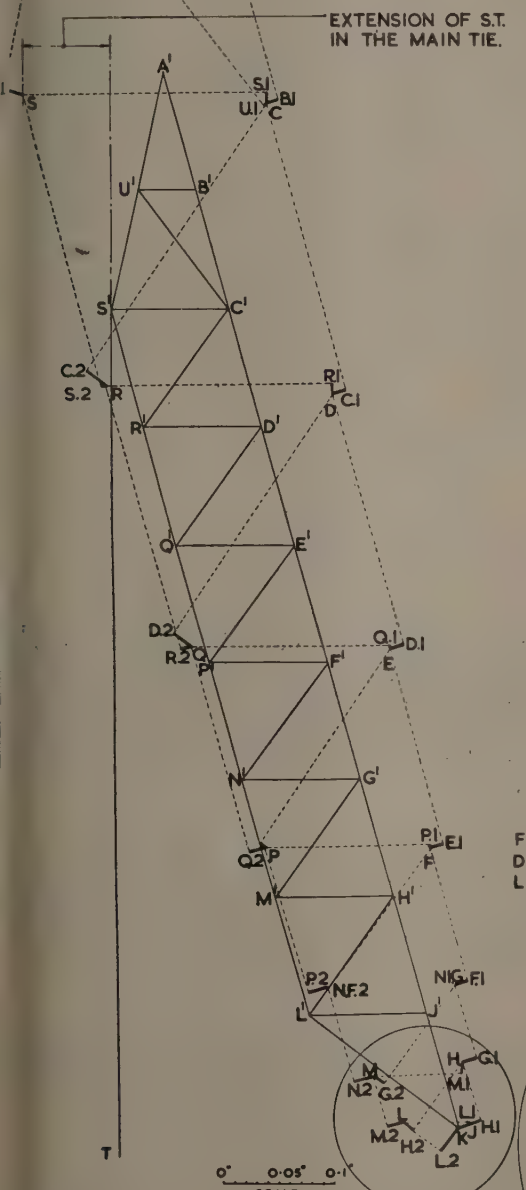


FIG. 3.

WILLIOT MOHR DIAGRAM FOR 117'-0" SPAN TIED ARCH.
FOR WILLIOT DIAGRAM ASSUME 'K' FIXED IN POSITION, JK FIXED IN DIRECTION.
FOR MOHR CORRECTION ROTATE ABOUT 'K' GIVING 'ST' NO ROTATION SO THAT HORIZONTAL DEFLECTION OF 'K' EQUALS HORIZONTAL DEFLECTION OF 'T' i.e. ZERO.

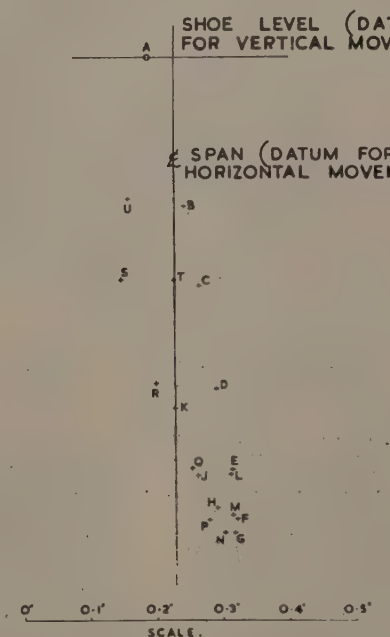
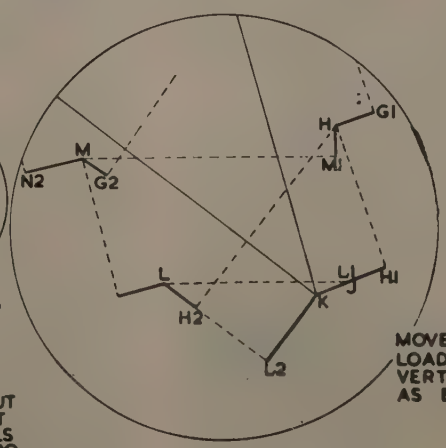


FIG. 4.
DIAGRAM SHEWING VERTICAL & LATERAL MOVEMENTS OF NODE POINTS UNDER DEAD LOAD.

	VERTICAL MOVEMENT INCHES	HORIZONTAL MOVEMENT INCHES
A	·535↑	·04←
B	·306↑	·014←
C	·185↑	·036←
D	·029↑	·064←
E	·095↓	·084←
F	·170↓	·092←
G	·190↓	·090←
H	·153↓	·061←
J	·103↓	·032←
K	–	–
L	·100↓	·083→
M	·165↓	·086→
N	·190↓	·074→
P	·169↓	·050→
Q	·091↓	·024→
R	·038↑	·029→
S	·195↑	·080→
T	·195↑	–
U	·319↑	·069→

MOVEMENTS TABULATED FOR DEAD LOADS ONLY WITH 'K' AS BASIS.
FOR TOTAL MOVEMENT i.e. DEAD + SERVICES + SNOW MULTIPLY BY 2·14



	VERTICAL MOVEMENT INCHES	HORIZONTAL MOVEMENT INCHES
A		·09←
B	0·49↑	·03←
C	0·74↑	·08←
D	1·08↑	·14←
E	1·34↑	·18←
F	1·50↑	·20←
G	1·54↑	·19←
H	1·47↑	·13←
J	1·36↑	·07←
K	1·14↑	–
L	1·35↑	·18→
M	1·49↑	·18→
N	1·55↑	·16→
P	1·50↑	·11→
Q	1·33↓	·05→
R	1·06↑	·06→
S	0·72↑	·17→
T	0·72↑	–
U	0·46↑	·15→

MOVEMENTS TABULATED FOR TOTAL LOADS WITH 'A' AS BASIS FOR VERTICAL MOVEMENT, ℄ OF SPAN AS BASIS FOR HORIZONTAL MOVEMENT

Figures 3, 4 and 5

Load Summations in Truss Members. Table No. 1

	1	2	3	4	5 (a & b)	7 & 8	Case I	Case VI
Member	Dead (tons)	Snow (tons)	Services (tons)	Suction (tons)	Wind Thrust and drag (tons)	Thermal Rise or fall. (tons)	(1) + (2) + (3)	(1) + (2) + (3) + (8)
B.1	+19.5	+13.5	+ 8.6	—13.7	+ 2.2	± 1.2	+41.6	—
C.2	+19.5	+13.5	+ 8.6	—13.8	+ 2.2	± 1.2	+41.6	—
D.5	+22.0	+15.3	+ 9.7	—16.2	+ 1.9	± 1.0	+47.0	—
E.7	+24.5	+17.0	+10.8	—18.3	+ 1.7	± 0.9	+52.3	—
F.9	+25.3	+17.5	+11.2	—19.2	+ 1.5	± 0.8	+54.0	—
G.11	+25.3	+17.5	+11.2	—19.2	+ 1.3	± 0.7	+54.0	—
H.13	+23.5	+16.3	+10.4	—18.3	+ 1.1	± 0.6	+50.2	—
J.15	+20.8	+14.4	+ 9.2	—16.6	+ 0.9	± 0.5	+44.4	—
D.16	+20.8	+14.4	+ 9.2	—16.9	+ 0.9	± 0.5	+44.4	—
N.1	—19.5	—13.5	— 8.6	+13.4	+ 2.5	± 1.3	—41.6	—42.9
N.3	—18.8	—13.0	— 8.3	+13.0	+ 2.5	± 1.3	—40.1	—41.4
M.4	— 2.0	— 1.4	— 0.9	+ 1.9	— 1.4	± 0.7	— 4.3	—
M.6	— 5.0	— 3.5	— 2.2	+ 4.3	— 1.2	± 0.6	—10.7	—
M.8	— 7.6	— 5.3	— 3.4	—	— 1.0	± 0.5	—16.3	—
M.10	— 8.4	— 5.8	— 3.7	+ 6.8	— 0.8	± 0.4	—17.9	—
M.12	— 8.4	— 5.8	— 3.7	+ 6.8	— 0.6	± 0.3	—17.9	—
M.14	— 6.5	— 4.5	— 2.9	+ 5.6	— 0.4	± 0.2	—13.9	—
M.16	— 6.4	— 4.5	— 2.8	+ 5.5	— 0.3	± 0.1	—13.7	—
M.N.	—16.5	—11.5	— 7.3	+10.9	+ 3.8	± 2.0	—35.3	—37.3
3-4	+ 5.3	+ 3.7	+ 2.3	— 3.8	— 0.23	± 0.12	+11.3	—
9-10	+ 0.6	+ 0.4	+ 0.3	— 0.5	— 0.23	± 0.12	+ 1.3	—
15-16	+ 1.7	+ 1.2	+ 0.8	— 1.4	—	—	+ 3.7	—
1-2	+ 0.6	+ 0.4	+ 0.3	— 0.4	—	—	+ 1.3	—
5-6	+ 3.0	+ 2.1	+ 1.3	— 2.3	— 0.23	± 0.12	+ 6.4	—
7-8	+ 2.4	+ 1.7	+ 1.1	— 1.9	— 0.23	± 0.12	+ 5.2	—
11-12	—	—	—	—	— 0.23	± 0.12	—	—
13-14	— 1.8	— 1.3	— 0.8	+ 1.4	— 0.23	± 0.12	— 3.9	—
2-3	— 0.7	— 0.5	— 0.3	+ 0.5	—	—	— 1.5	—
4-5	— 3.8	— 2.7	— 1.7	+ 2.9	+ 0.27	± 0.14	— 8.2	— 8.34
6-7	— 3.2	— 2.2	— 1.4	+ 2.3	+ 0.27	± 0.14	— 6.8	— 6.94
8-9	— 0.8	— 0.6	— 0.4	+ 0.6	+ 0.27	± 0.14	— 1.8	— 1.94
10-11	—	—	—	—	+ 0.27	± 0.14	—	— 0.14
12-13	+ 2.3	+ 1.6	+ 1.0	— 1.7	+ 0.27	± 0.14	+ 4.9	—
14-15	+ 3.0	+ 2.1	+ 1.3	— 2.3	+ 0.27	± 0.14	+ 6.4	—

Design cases II, III, IV and V were less than Case I, with no reversals.
Case VI gave increased tensions in the members noted.

in truss members are summarised in Table No. 1, and truss member sizes together with a calculation for spread under dead load, are shown in Table No. 2.

A Williott-Mohr diagram, reproduced in Fig. 3 shows the vertical deflection and spread, and the calculated dead load spread of .086 in. compared with the spread of .08 in. given by the graphical solution.

Movements of the various node points relative to the ridge and shoe are set out in Fig. 4, the deflected shape of the truss under dead + snow + services is shown diagrammatically in Fig. 5 with an enlarged scale for the vertical movements for purposes of clarity, and details of the truss are shown in Fig. 6.

Rafters.

Rafters were designed to accord with Table 7 of BSS.449 (1948) taking an effective length for the rafters about the Y-Y axis as 0.8 of the distance between longitudinal girders which were also the intersection points of the rafter bracings.

Main Tie.

The assumptions made in producing the Williot-Mohr diagram pre-supposed that the main tie remained horizontal under vertical load. The introduction of droppers between the latticed rafter system and the main tie, however, induced variations of movement in the main tie caused by relative deflections of te node points on the lines of the droppers. This lo deformation in the tie was treated as a case analog to that of relative settlement of supports. The duced bending moment results in a stress of 1.8 *T in.* which was combined with the axial stresses in te main tie.

Horixontal Sling.

Under full vertical loads the lateral movement point L on the truss towards the centre line amou to 0.18 in. The horizontal sling introduced at t level was therefore provided with slotted holes accommodate twice this movement.

Shoes.

Shoe details were designed to accommodate rafter and tie loads, and the gusset, which was exten above the top of the rafter back, was so proportio to keep the stresses below 9 tons per sq. in.

Roller Shoes.

In view of the need to reduce friction and to k induced compressive forces in the trusses to a m mum, transverse thermal and wind movement catered for by the provision on every third line trusses of a greased roller shoe. Design of this based upon a permissible load of 0.5d tons per in. on mild steel rollers where d represents the diame

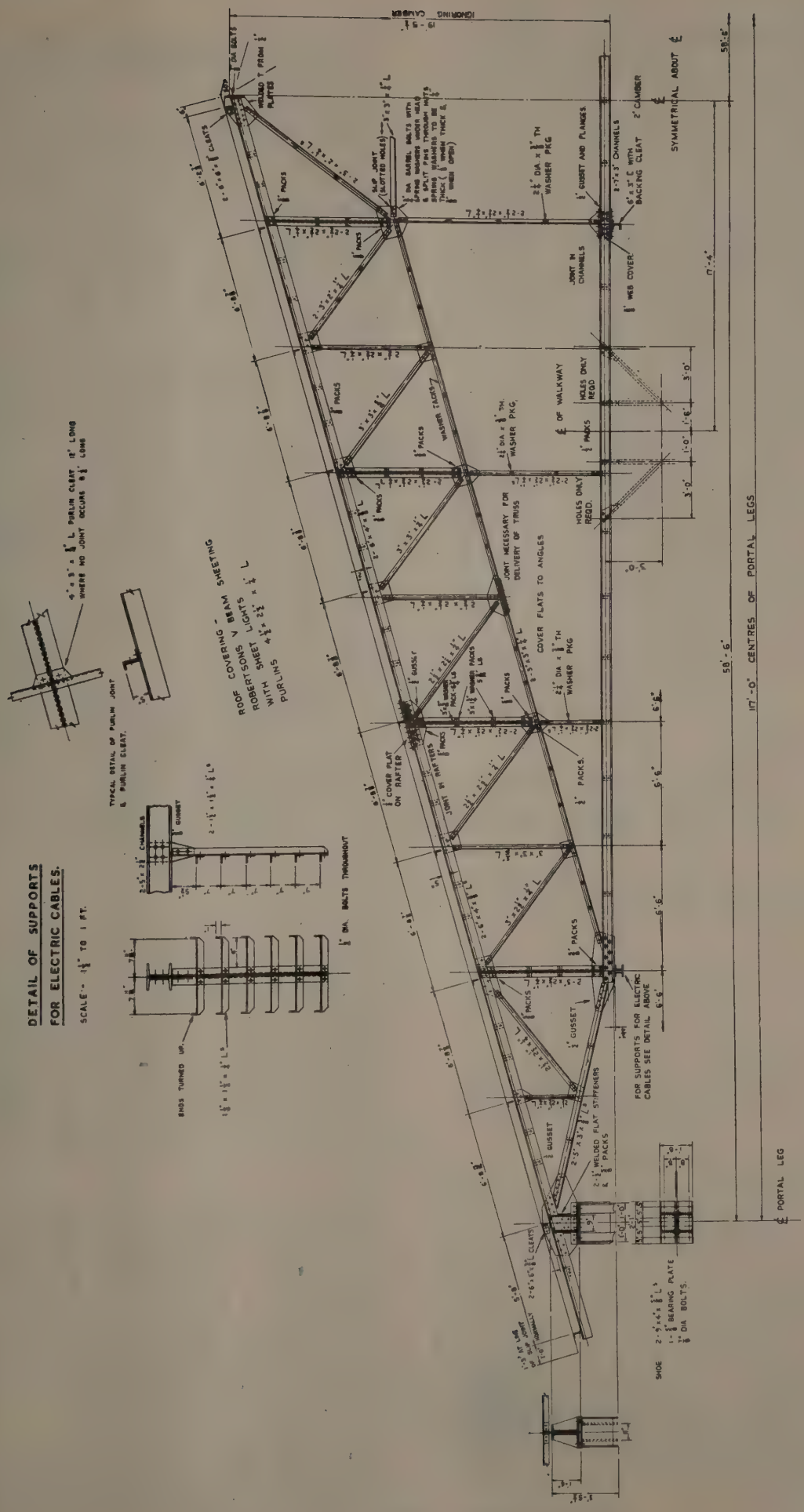


Fig. 6.—117 ft. 0 ins. Span 3 pinned Tied Arch

MEMBER	F	U	SECTION	L/A			FL/A	FUL/A+	FUL/A−	U²L/A	FL/A
B-1	+19·5	+0·48		81/11·72	=	6·9	+135·0	64·5		1·58	−·01
C-2	+19·5	+0·48				6·9	+135·0	64·5		1·58	−·01
D-5	+22·0	+0·43				6·9	+152·0	65·5		1·28	−·01
E-7	+24·5	+0·38				6·9	+169·0	64·0		0·99	−·01
F-9	+25·3	+0·34	2/6" × 4 × 5/8" L's.			6·9	+174·0	59·1		0·80	−·01
G-11	+25·3	+0·29				6·9	+174·0	50·5		0·58	−·01
H-13	+23·5	+0·24				6·9	+162·0	39·0		0·40	−·01
J-15	+20·8	+0·20				6·9	+143·0	28·6		0·28	−·01
K-16	+20·8	+0·20				6·9	+143·0	28·6		0·28	−·01
N-1	−19·4	+0·56	2/5" × 3" × 3/8" L's.	81/5·72	=	14·2	−276·0		154·0	4·45	+·02
N-3	−18·8	+0·56				14·2	−267·0		150·0	4·45	+·02
M-4	− 2·0	−0·31		81/2·88	=	28·1	− 56·2	17·5		2·70	+·00
M-6	− 5·0	−0·26				28·1	−140·5	36·5		1·90	+·01
M-8	− 7·6	−0·22	2/3" × 3" × 1/4" L's.			28·1	−214·0	47·0		1·36	+ 01
M-10	− 8·4	−0·18				28·1	−236·0	42·5		0·91	+·01
M-12	− 8·4	−0·13				28·1	−236·0	30·7		0·47	+·01
M-14	− 6·5	−0·08				28·1	−183·0	14·6		0·18	+·01
M-16	− 6·4	−0·06	2/3" × 2" × 1/4" L's.	126/2·38	=	53·0	−340·0	20·4		0·19	+·02
M-N	−16·5	+0·84	2/7" × 3" L's.	45·5×12/8·36	=	65·3	−1080·0		910·0	46·00	+·08
3-4	+ 5·3	−0·05	2/3" × 2 1/2" × 1/4" L's.	80/2·62	=	30·5	+162·0		8·1	0·08	−·01
9-10	+ 0·6	−0·05	2 1/2" × 2 1/2" × 1/4" L	80/1·19	=	67·1	+ 40·3		2·0	0·17	−·00
15-16	+ 1·7	−	2/2 1/2" × 2 1/2" × 1/4" L's.	80/2·38	=	33·5	+ 57·0				
1-2	+ 0·6	−	2 1/2" × 2 1/2" × 1/4" L	40/1·19	=	33·5	+ 20·0				−·00
5-6	+ 3·0	−0·05	3" × 3" × 5/16" L	80/1·78	=	44·9	+134·7		6·75	0·11	−·01
7-8	+ 2·4	−0·05	2/2 1/2" × 2 1/2" × 1/4" L's.	80/2·38	=	33·5	+ 80·5		4·02	0·08	−·00
11-12	0	−0·05				33·5	−			0·08	
13-14	− 1·8	−0·05	2 1/2" × 2 1/2" × 1/4" L's	80/1·19	=	67·1	−121·0	6·05		0·17	+·00
2-3	− 0·7	−				67·1	− 47·0				+·00
4-5	− 3·8	+0·06	3" × 2 1/2" × 1/4" L	97/1·31	=	74·0	−281·0		16·9	0·27	+·02
6-7	− 3·2	+0·06	2 1/2" × 2 1/2" × 1/4" L	97/1·19	=	81·5	−262·0		15·7	0·30	+·02
8-9	− 0·8	+0·06				81·5	− 65·0		3·9	0·30	+·00
10-11	0	+0·06	3" × 3" × 1/4" L	97/1·44	=	67·2	−			0·24	
12-13	+ 2·3	+0·06	3" × 3" × 5/16" L	97/1·78	=	54·4	+125·5	7·5		0·19	−·00
14-15	+ 3·0	+0·06	2/3" × 2" × 1/4" L's	126/2·38	=	53·0	+159·0	9·6		0·19	−·01
						TOTAL FOR 1/2 TRUSS.		696·65	1271·37	72·56	
						OTHER HALF		696·65	1271·37	72·56	
								1393·3	2542·74	145·12	

TABLE No.2.

SPREAD UNDER DEAD LOAD = $\frac{2542 \cdot 74 - 1393 \cdot 3}{13,400}$ = ·086"

SHORTENING UNDER 1 TON THRUST = $\frac{145 \cdot 12}{13,400}$ = ·0109"

Table No. 2

of the rollers in inches. The frictional co-efficient was taken as 5 per cent. although BSS.153 specified 3 per cent.

Ridge.

The bolted connection at the ridge was designed to resist the shear due to snow conditions on one roof slope only.

Riveting versus Welding.

The decision to use a riveted design for the truss rather than a welded one may be worth recording at this stage.

It was foreseen that within a few weeks of the start of erection of the building, the erection rate on the site would rise steeply to approximately 950 tons of relatively light structural steelwork per month for a period of some months. This would absorb most of the capacity of the Constructional Workshops to produce the necessary steel in the appropriate order even if a balance were struck as between riveted and welded work. It therefore became necessary to choose as between riveted trusses and welded portals or vice versa. The decision to use the former alternative was largely dictated by the consideration that welded portals would provide more flexibility for the designer and also that welded lacings would probably cater more effectively than would riveted, for the rather high elastic shears which were likely to be developed in the portal members.

Rafter Bracings.

BSS.449, paragraph 51a lays down a basis of design for the bracings of laced strut members. For this purpose an arbitrary transverse shear force amounting to 2½ per cent. of the axial load in the compression member is applied at any point in its length and the lacings are designed to resist this shear force at normal stresses. For design of the rafter bracings, a figure of 2½ per cent. of the maximum load in the rafter was taken and multiplied by the number of rafters stabilized by the Bracing System. The ability of the purlins to transmit these forces was checked and accepted with an allowable over stress of 16.2/3 per cent.

Longitudinal Bracings.

These were designed to restrain the latticed rafter system of the truss and in addition to carry the intermediate rafter.

Longitudinal Slip Joints.

The slip joints along the roof of the building take the form of slotted holes in the purlins. Shoulder bolts with split pins were provided to minimize the effect of friction.

Gable Framing.

In the early days of design it was known that use would be made of the gable framing posts for carrying cooling water mains and 11 K.V. cables,

hough little was known of these in detail. In dition there was always the possibility that another ilding with perhaps cranage requirements would entually be attached to the Process Building. The anes of course, could be electric travelling gantries pported upon girders bracketed off the framing sts, or blocks slung from the roof trusses of the ture building.

Where information on such matters was incomplete, became a general practice to design the gables for eir known general conditions, and as an alternative, include for supporting the shoe of a future 40 ft. an building 30 ft. high to eaves, with provision, addition, for a crane bracket to carry a 5 ton O.E. ane.

The extension usually materialised, but in the latter ages of the project the span turned out to be 80 ft. in. with 1½T or 3T hand operated blocks slung from e roof trusses.

Cables were brought into the Building through derground ducts and carried up the gable framing, ong the gable, into and along the bays at truss tie vel, progressively peeling off to the transformers the cell tops.

Corridor Portals

eneral.

Headroom from floor level to the lower beam carry- g the transformer platform and clearances between per and lower beams were laid down by the plant signer who also tied down the overall dimensions of e columns.

Several structural shapes were considered for this amework, but open laced welded systems for the embers were eventually adopted, to give the best ssible form of adjustment on both axes to meet the al requirements of deformation and design stresses cluding those due to thermal movement, and at the me time keep the weight of steel to a minimum.

esign.

As already mentioned, the stability of the building gainst horizontal forces, whether from wind or tem- erature was based upon the interdependence of three rtals in line transversely, and the ability of the of to transmit such forces to the portals, combined ith the ability of the portal system along the corri- rs to deal with thermal movements in that direction. hese induced horizontal forces, in combination with e worst likely combination of vertical loads required careful analysis.

It was not possible to achieve any degree of fixity at the foot of the portals due to the presence of sub-ground cooling water return mains along the corridors on the one side, and the proximity of the cell floor slabs on the other.

The greatest effects of transverse wind and drag, plus expansion, were likely to occur in Portal ' C ' as shown in Fig. 7 as the bending stresses from these causes could be cumulative. It will be understood however, that with the various additional alternative conditions of loadings from gutter, crane, transformer platform, etc., ' envelope ' moments were necessary to cover the worst conditions for each of the portal members for more than one portal, in order to determine the critical design conditions for each, and although every portal would not necessarily be subject to the most severe stresses it was decided to make them all similar, not only to satisfy the original design assumptions, but to facilitate jigging in the shops.

Calculations showed that deformation of the truss under unit horizontal thrust was only .011 in., so initial portal design ignored this shortening. It will be seen later, that when truss shortening is taken into account it provides a slight easement to the final movement and stresses in the portal under consideration.

Apart from vertical loads from roof trusses, which produce no bending in the portals, the following design conditions were analysed :

Vertical loads—

A—Gutter.
B—Crane under varying working conditions.

Transverse loads—

C—Portal ' C ' with thermal movement, transverse wind and drag, and truss deformation under wind, suction and drag.

Longitudinal Forces—

D—Thermal movement along the corridor.

Evaluation of the bending moments for Conditions ' A ' and ' B ' presented no problems but adjustments for side sway were necessary in the latter case due to unequal crane reactions. For this particular case however, a centrally loaded crane is taken.

For Condition ' C ' wind and temperature effects were naturally dealt with separately. Temperature movement takes place in Portal ' C ' relative to the centre line of Portal ' B ' and is equal to 0.64 in. for a variation of 60°F. Movement due to wind and drag must therefore not exceed 1.5 in.—0.64 in.=0.86 in.

The design condition for wind is as follows :

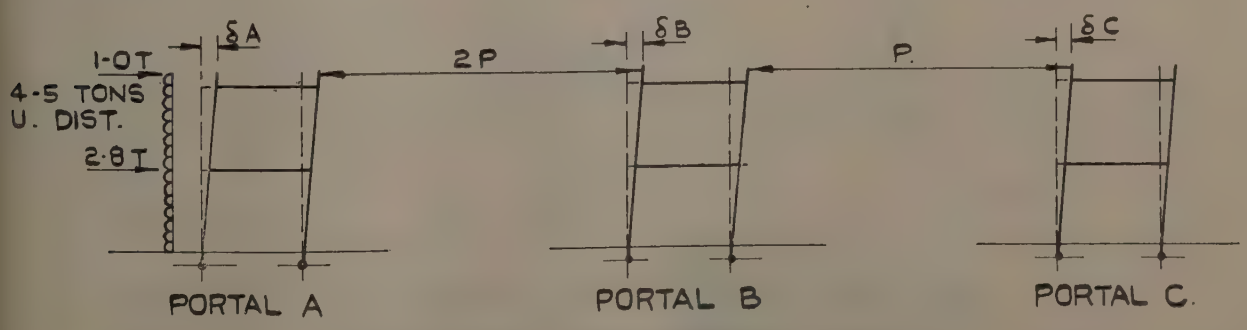


Fig. 7.—Portals.—Design Conditions for Wind

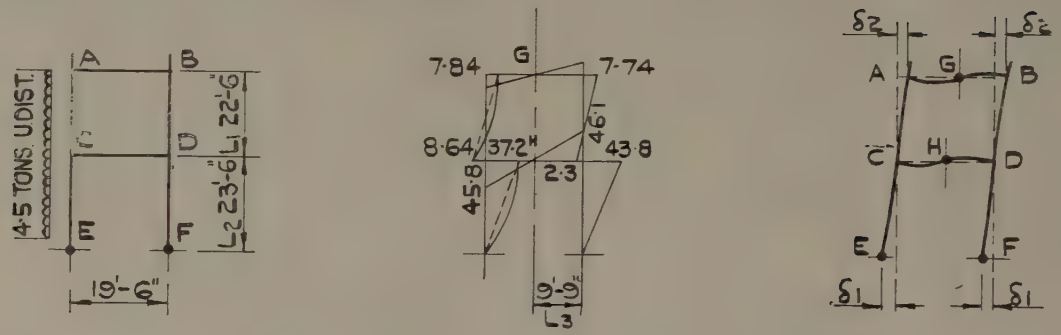
and assuming no truss shortening it follows that $\delta A = \delta B = \delta C$.

Rough initial calculations of loads and moments were made and the following frame inertias were assumed :

Column Ixx = 2430 in.^{4}
Column Iyy = 1020 in.^{4}
Top Beam = 842 in.^{4}
Lower Beam = 2533 in.^{4}

The results are summarised below :

i. *Due to a lateral load of* 4.5 *tons uniformly distributed.*

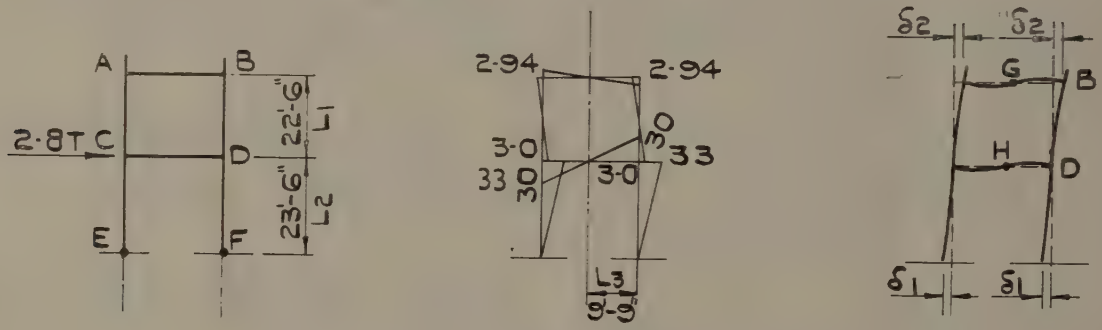


$$\delta_1 + \delta_2 = 0{\cdot}61'' + 0{\cdot}158'' = 0{\cdot}768''.$$

Fig. 8.—Portals.—Deformation under lateral load of 4.5^T uniformly distributed

$\delta_1 + \delta_2 = 0.61$ in. $+ 0.158$ in. $= 0.768$ in.

ii. *Due to lateral load of* 2.8 *tons applied at C.*

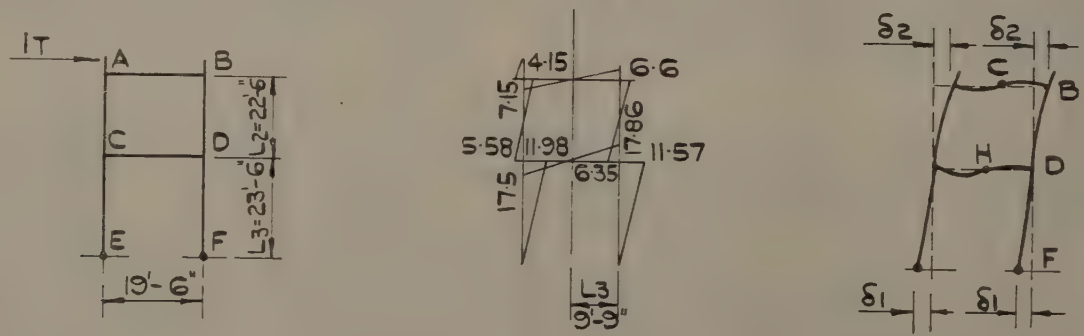


$$\delta_1 + \delta_2 = 0{\cdot}438'' + 0{\cdot}125'' = 0{\cdot}563''$$

Fig. 9.—Portals.—Deformation under lateral load of 2.8^T applied at C

$\delta_1 + \delta_2 = 0.438$ in. $+ 0.125$ in. $= 0.563$ in.

iii. *Due to lateral load of* 1.0 *tons applied at top.*



$$\therefore \text{ FOR 1 TON THRUST. } \delta_1 + \delta_2 = 0{\cdot}183'' + 0{\cdot}092'' = 0{\cdot}275''.$$

Fig. 10.—Portals.—Deformation under lateral load of 1.0^T applied at top of portal

$\therefore$ for 1T thrust,

$\delta_1 + \delta_2 = 0.183$ in. $+ 0.092$ in. $= 0.275$ in.

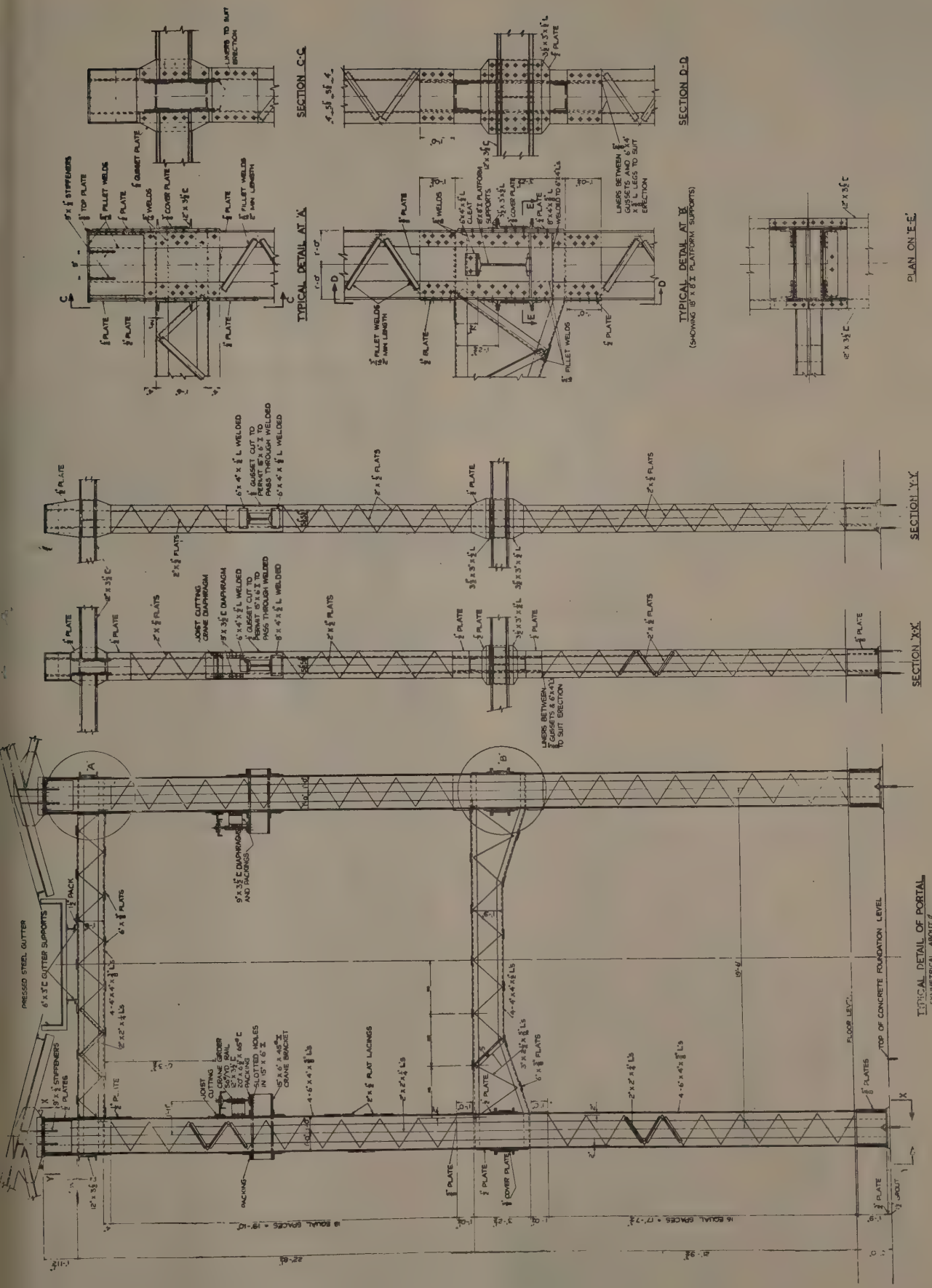


Fig. 11.—Details of Portal

From Condition iii it follows that a load of P^T applied at the top of the portal will cause a deformation of 0.275 P in.

Hence referring to Fig. 7 and ignoring truss deformation it follows that :

$$\delta A = \delta B = \delta C = 0.275P$$
but $\delta A = 0.275\ (1\text{-}2P) + 0.768 + 0.563$

$$\therefore\ 0.275P = 0.275 - 0.550P + 0.768 + 0.563$$

Hence $P = 1.95$ tons

$$\delta C = \delta B = \delta A = 0.275 \times 1.95 = 0.535 \text{ in.}$$

Hence wind deformation for Portal C
$= 0.535$ in.

and deformation due to drag
$= 0.6T \times 0.275$ in. $= 0.16$ in.

Maximum wind thrust in truss
$2P = 3.9$ tons.

Table No. 3 summarises the lateral movements in Portal C from truss loads, temperature and wind, including the associated effects of drag, suction, etc. on the roof.

Table No. 3. Deformation of Portal 'C'

Summary of Worst Combination of Circumstances causing Tranverse Movements in Portal 'C'

i	Spread of truss due to dead load (18 lbs./sq. ft.)	0.086 in.	—
ii	Spread of truss due to full services (8 lbs./sq. ft.)	0.038 in.	—
iii	Thermal expansion of truss + half width of corridor	0.640 in.	—
iv	Wind thrust and drag 0.535 in. + 0.16 in.	0.695 in.	—
v	Shortening of truss due to suction = 0.5 (0.03 + 0.06)	—	0.045 in.
vi	Lengthening of truss due to drag	0.004 in.	—
vii	Shortening of truss due to wind thrust .011 (3.9 + 1.95) (.5) ..	—	0.032 in.
	TOTALS :	1.463 in.	0.077 in.

Net lateral deformation of Portal C = 1.463 in. — 0.077 in = 1.386 in.

But 1 ton thrust at top of portal causes a deformation of 0.275 in.

$\therefore$ a deformation of 1.385 in. represent a thrust of $\frac{1.385}{0.275} = 5.0T$ at the top of the portal, which is the thrust used in the design of Portal C, the resultant bending moments due to this force being tabulated against condition C in Table No. 4. The nett lateral deformation due to temperature and dead loads only (i and ii and iii) = +0.764 in., which is equivalent to a thrust of 2.78T at top of portal.

D. *Thermal expansion along the corridors.*

Expansion on a length of 78 ft. 9 in. due to 60°F temperature rise = 0.000007 in. × 78.75 × 12 × 60
= 0.40 in.

The maximum bending moment at joint FA due t temperature expansion was calculated to be approx mately 12 ft. tons.

This bending moment together with the envelop moments in Table No. 4 and a maximum vertical loa of 61 tons in the leg of the portal provide the desig conditions for portal C, details of which are shown i Fig. 11.

Because of headroom, the shape of the central bea had to be modified in order to cater for the very hig bending moments developed at the junction with t columns. A check calculation on this revised shap showed a variation in moments of less than 5 per cen so the adjusted section was accepted.

PHASE 2 SUPERSTRUCTURE

This took the form of an extension at the northe end of Phase 1.

The general proportions of the building permitte an approach in design similar to that for Phase 1, b by now it was considered that the working temperatu conditions for personnel were insufficiently unpleasa to justify the provision of any large scale ventilatio plant, and that relaxations on movements in the stru ture could be accepted as well.

A more normal type of roof truss could according be considered and as a consequence, a modified Fin truss was adopted, details of which are shown in Fi 12. These trusses were spaced at 22 ft. 6 in. centre and not only did this arrangement reflect a saving 15 per cent. in the weight of roof steel, but it speede up enormously the rate of erection.

Apart from this change, the general framewo continued on similar lines to that for Phase 1, exce for minor changes in detail to accommodate variatio in the requirements for carrying cables and services.

The siting of the Phase 2 Sub-Station had not be settled at the time Phase 2 superstructure was bein designed, so it was not known where the 11 K.V. cabl would enter the building or through how many unde ground ducts. The normal gable framing posts we 18 in. × 6 in. R.S.J.'s, but these were inadequate deal with the wide banks of rising cables from t cable ducts as well as the loads from cooling mai etc., so to maintain an 18 in. depth of post (for sta dardization of bracket details etc.) 18 in × 7 R.S.J.'s were selected for this purpose. One full b of these heavier framing posts was therefore order arbitrarily, and the position, details etc. were sub quently adjusted after the cable entries had be finalised.

Table No. 4

Tranverse Moment Summation Tables for Portal C in ft. tons:—

Conditions	AB	AC	BA	BD	CA	CD	CE	DB	DC	DF
A Gutter	— 5.62	+ 5.62	+ 5.62	— 5.62	+ 1.76	— 0.85	— 0.90	— 1.76	+ 0.85	+ 0.9
B Crane	— 1.29	+ 1.29	+ 1.29	— 1.29	+ 4.91	— 2.52	— 2.42	— 4.91	+ 2.52	+ 2.4
C 5T thrust	+35.75	—20.75	+33.0	—33.0	—27.9	+87.5	—59.9	—31.75	+ 89.3	—57.85
ENVELOPE B.M.S.	+35.75	—20.75	+39.91	—39.91	—27.9	+87.5	—63.22	—38.42	+92.57	—57.85

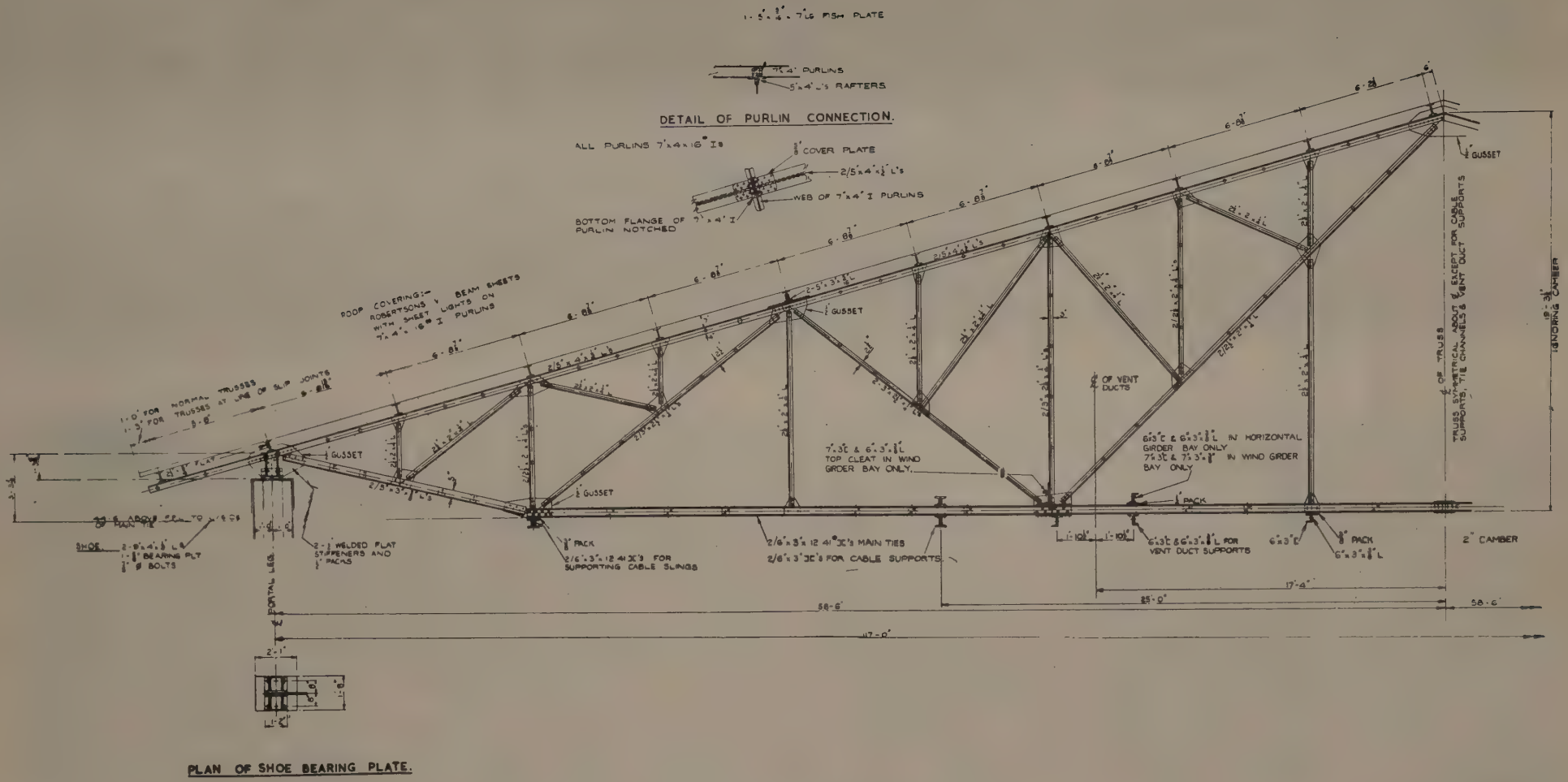

DETAIL OF PURLIN CONNECTION.
ALL PURLINS 7"x4"x16" Is
COVER PLATE
2/5"x4"x½" L's
WEB OF 7"x4" I PURLINS
BOTTOM FLANGE OF 7"x4" I PURLIN NOTCHED
ROOF COVERING:- ROBERTSONS ½" BEAM SHEETS WITH SHEET LIGHTS ON 7"x4"x16" I PURLINS
GUSSET
PACK
2/6"x3"x12.41" JC's MAIN TIES
2/6"x3" JC's FOR CABLE SUPPORTS
2/6"x3"x12.41" JC's FOR SUPPORTING CABLE SLINGS
7"x3" C & 6"x3"x½" L TOP CLEAT IN WIND GIRDER BAY ONLY.
6"x3" C & 6"x3"x½" L IN HORIZONTAL GIRDER BAY ONLY
7"x3" C & 7"x3"x½" IN WIND GIRDER BAY ONLY
6"x3" C & 6"x3"x½" L FOR VENT DUCT SUPPORTS
2" CAMBER
SHOE
2-8"x4"x½" L's
1-1" BEARING PLT
1" ø BOLTS
2-½" WELDED PLAT STIFFENERS AND ½" PACKS
PORTAL LEG
58'-6"
117'-0"
PLAN OF SHOE BEARING PLATE.

Fig. 12.—117 ft. 0 ins. Span Modified Fink Truss

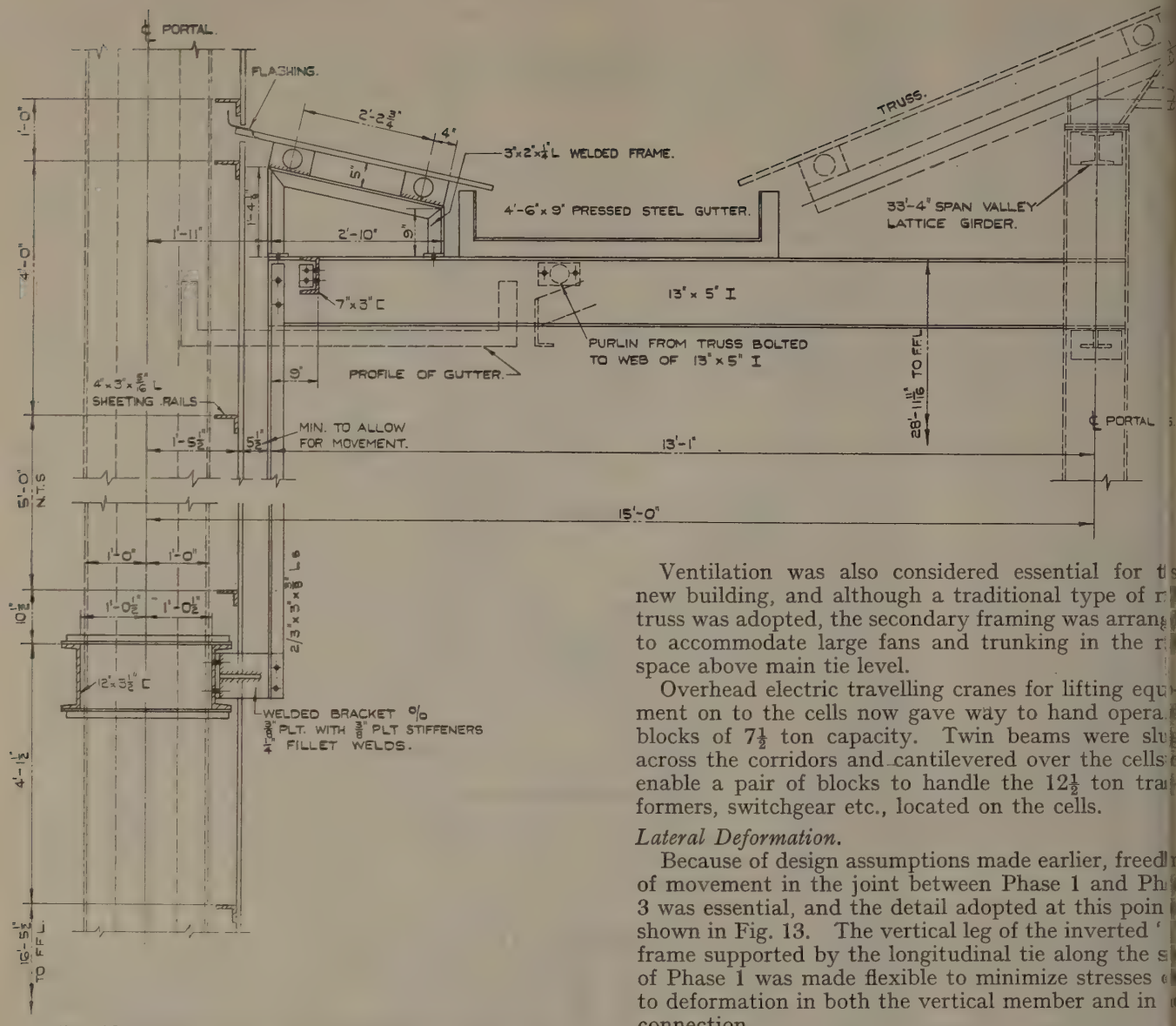


Fig. 13.—Flexible Joint between Phases 1 and 3

The services carried by the normal gable framing amounted in some instances to 4/18 in. diameter cooling mains weighing 0.27 tons per ft. run, plus a bank of cables weighing 0.43 tons per ft. run.

PHASE 3. SUPERSTRUCTURE

General.

The size of the cells was now greatly reduced thus affecting the height and the gridding of the new building which was made 30 ft. 0 in. high to eaves, having roof spans of 75 ft. 0 in. and 65 ft. 0 in. with corridors 15 ft. 0 in. wide, and based on a general stanchion gridding of 26 ft. 9 in. along the corridors.

The decision to extend in a southerly direction from Phase 1, with a building 30 ft. 0 in. high to eaves, posed the question of how to attach it to the parent structure, which had been detailed to accommodate a future building of similar height.

Factory production had already started in the first building and structural connections to it had to be made from the outside, with an overriding requirement that ingress of rain or dust should be kept to a minimum.

Ventilation was also considered essential for t s new building, and although a traditional type of r truss was adopted, the secondary framing was arran to accommodate large fans and trunking in the r space above main tie level.

Overhead electric travelling cranes for lifting equ ment on to the cells now gave way to hand opera blocks of 7½ ton capacity. Twin beams were slu across the corridors and cantilevered over the cells enable a pair of blocks to handle the 12½ ton tra formers, switchgear etc., located on the cells.

Lateral Deformation.

Because of design assumptions made earlier, freed of movement in the joint between Phase 1 and Ph 3 was essential, and the detail adopted at this poin shown in Fig. 13. The vertical leg of the inverted ' frame supported by the longitudinal tie along the s of Phase 1 was made flexible to minimize stresses to deformation in both the vertical member and in connection.

A further problem was created by the fact t vertical wind bracings could not be permitted al the corridors. In fact, no horizontal structural me ber was acceptable below the level of the lifting bea referred to earlier.

The absence of a ' middle rail ' in the portals for t building increased their flexibility very considera and three portals were no longer able to keep w movements within reasonable limits. In fact, ma mum transverse deformation under such conditi was estimated as 2.0 in. In view of this, five port in line laterally, linked by the trusses were desig to share the lateral loads from wind and drag in c junction with their appropriate stresses due to ther movements, such movement being accommodated the flexible connection adjacent to Phase 1.

Longitudinal Deformation.

Deformation due to a longitudinal wind force resisted by a unit approximately 180 ft. 0 in. long likely to be in the order of 4.4 in., but as the roof intended to carry not only ventilation fans and tru ing, but also considerable quantities of cables and p services at truss tie level, dropping to equipment the chequer plate deck at cell roof level, such mo ment as between adjacent units was considered cessive. Furthermore, at the junction between

and new structure, severe and undesirable racking esses due to differential movements between the o buildings would be set up in the supporting mem- s of the corridor roof. It was therefore decided restrict the movement in the longitudinal expansion nts to ± 1 in., the slip joint in principle, still being ilar to that for Phase 1.

Assuming that the expansion joint is set initially its mean position and at a temperature which is the an of its range, then under hot conditions the joint ll have closed by 0.52 in. leaving a gap of 0.48 in. be closed by the wind acting on one unit, after ich the second unit would act with the first. Under ch conditions the maximum wind movement was imated to be approximately 2.50 in. Under cold nditions, the maximum movement from the original sition could be approximately 3.0 in.

But while the flexible nature of the corridor supports eated concern over possible wind movements, stresses t up by thermal movements were correspondingly all.

russes.

Now that the buildings were beginning to take shape d experience of factory requirements were being ined, and because of the need to economise in steel herever possible, it was decided to use tubular con- ruction for the roof trusses. In view of the fact at they are carried upon elastic supports, the process design followed a pattern similar to that for Phases and 2. Stress reversals in tension members however, ere never likely to be a critical feature governing sign.

The tubular trusses were designed in accordance ith the approved code for tubular steelwork, which that time had been issued for comment, and the ubular construction showed a paper saving of 40 per nt. when compared with the weight of steel in hot olled riveted trusses. Certain tube sizes and thick- esses were, however, unobtainable in time to suit ne erection programme, and substitutions to the earest suitable available tubes reduced this saving something like 33 per cent. The trusses were elivered to site in halves, "sleeve" type joints in ne main tie being welded up on the ground prior to fting.

Because of the need to seal the ends of tubes, shoe etails for tubular trusses tend to lose the simplicity hich is characteristic of tubular framework, and there- ore becomes an expensive item in the fabrication of usses.

Portals.

The corridor steelwork and roof supports were for easons already mentioned, designed as portals to ransmit lateral and longitudinal forces to the founda- ions. Once again fixity at the feet was not possible, ue to the presence of subground cooling water return ains along the corridors on the one side and the roximity of cell floor slabs on the other.

CELL SUPERSTRUCTURES

Phase 1.

General.

The Plant, consisting of a number of stage units vith interconnecting pipework, cabling, etc., is housed n cells, each approximately 54 ft. 0 in. × 43 ft. 0 in. × 20 ft. 0 in. high with an internal dry atmosphere nd a working temperature of 140°F. The walls and oof are cladded with an insulation having a thermal conductance U of 0.1 and sufficiently air tight to prevent ingress of moisture or air containing water vapour, to any extent likely to tax the capacity of the air drying plant, the duty of which is to maintain within the cell a dew point of 14°F., which at the working temperature of 140°F. is equivalent to a relative humidity of less than 2 per cent. Each cell has an air tight door 11 ft 0 in. × 9 ft. 0 in. high.

All electrical controls, transformers, switchgear, instrument panels, gas valves, etc. are operated from a chequer plate working deck on top of the cells, the space between the working deck and the roof insulation being utilised to accommodate cables and pipes. Process pipes pass through the wall cladding, connecting plant in adjacent cells. Air tight wall cladding is provided on the inside face of the stanchions, and thermal wall cladding on the outside, with an air space between for convection purposes. The roof insulation is attached to the underside of the roof beams and provides both thermal insulation and airtightness. Pipes and cables are slung from both the walls and the roof of the cells.

Design.

The cladding used for the cells for Phase 1, consisted of hollow aluminium panels 8 ft. × 4 ft. containing layers of aluminium foil, with stiffened channel edges. These panel sizes largely determined the grid of the framework, being self supporting over 8 ft. 0 in. span.

The depth of main beams of necessity had to be the same throughout, in view of the fact that the top flange supported a working deck of chequer plate, and the bottom flange the cladding. For economical main beam design, the heavy transformer and switchgear were located over or near a line of stanchions. It was then possible to use 18 in. × 7 in. and 18 in. × 6 in. R.S.J. main beams, spanning 43 ft. 0 in., limitations of deflection being the governing feature for design, and 8 in. × 6 in. and 8 in. × 4 in. R.S.J. stanchions at 8 ft. 0 in. centres.

The cells are grouped into blocks of 4, each block being designed as a stable unit. To control expansion, each line of stanchions was braced at the centre of its length, and stability of the block was achieved by the combination of this bracing and the chequer plate deck which acted as a horizontal diaphragm tying the tops of the stanchions together and spanning horizontally between the bracings. Erection bracing was provided at the corners of each block and removed upon completion to permit the cells to expand freely under thermal effects.

This arrangement, with the cladding outside the stanchions, and under the beams, means that the stanchions and mid-height tie beams are subject to high temperatures, whilst the roof beams including the eaves tie beams are not. The expansion of the mid-height tie beam is therefore greater than that of the eaves beam and as the average temperature of the floor slab is less than the cell temperature the central tie expands more than the slab. This movement gives rise to a bending stress of 4 tons per sq. in. in the corner stanchions and proportionately less in others depending upon their position relative to the central bracing.

Although the arrangement of wall cladding on the outside face of the steel was undesirable because of induced temperature stresses in the stanchions, it facilitated the provision of internal wall brackets for pipes and cables.

For carrying pipes inside the cells, a subsidiary layer of light beams was slung below the cladding, supported from plate hangers welded to the under side of the main beams and passing through joints between the cladding panels, the cladding cover strips being slotted to accommodate the plates. Where the subsidiary steelwork carried horizontal loads from pipe hangers the load was carried by plate anchors welded to the main beams.

Chequer plate was used for the working deck on top of the cells as this permitted a certain measure of flexibility to cover for possible changes of layout, for cutting holes, wells, etc., at a later date.

Phase 2.

General.

The requirements of the Plant Engineer in this case were similar to those for Phase 1, but with the following changes :

The size of the cells now became approximately 108 ft. × 43 ft. × 20 ft., grouped two per block, and the cladding was to provide a thermal conductance U of 0.2. The easement in the value of U now permitted the use of 1 in. thick aluminium faced cork which was available in panels up to 8 ft. × 4 ft. and capable of spanning 4 ft. The hollow wall construction used in Phase 1 was eliminated, so it was now possible to fix the cladding to the inside face of all steelwork, thus reducing temperature stresses in the stanchions and simplifying the cladding details at the junction of roof and wall. It did, however, create problems in the fixing of internal wall brackets for pipes and cables.

Secondary Steel.

Supports for the new type of cladding involved the use of additional secondary steel members at 4 ft. 0 in. centres, having a minimum flange face of 4 in. Secondary members in hot rolled sections would have involved a considerable weight of steel, so Cold Formed Sections in 8 gauge steel were adopted. The use of this new material raised several problems which had to be overcome to ensure satisfactory results, but these are referred to later.

For side cladding $4\frac{1}{2}$ in. × $1\frac{1}{4}$ in. × 8g. C.F. channel rails were used, for ceiling joists $4\frac{1}{2}$ in. × 3 in. × 1 in. × 8g. C.F. ⌐⌙ sections, and for eaves rails, 4 in. × 4 in. × 8g. C.F. angles.

To achieve the specified air-tight conditions, the meeting faces of rails with stanchions and/or beams had to be co-planar, and because of caulking requirements, gaps between edges of connected steelwork had to lie between limits of $\frac{1}{16}$ in. minimum and $\frac{3}{16}$ in. maximum. To achieve this degree of accuracy, the following fabrication and erection tolerances were specified :

Actual centres of adjacent stanchions and roof beams to be theoretical length — 0 in. + $\frac{1}{8}$ in. (overall growth of block each side of centre line not to exceed $\frac{3}{8}$ in.).

Actual length of C.F. steel member to be theoretical length + 0 in. — $\frac{1}{16}$ in.

The theoretical gap at meeting faces of members was detailed as $\frac{1}{16}$ in., which therefore became the minimum theoretical gap, and the maximum theoretical gap became $\frac{1}{16}$ in. + $\frac{1}{2}$ ($\frac{1}{8}$ in. × $\frac{1}{16}$ in.) = $\frac{5}{32}$ in.

Wherever possible, a web plate connection was used between secondary and main members to form a backing for " Prestik " mastic caulking. Where another type of connection was unavoidable, as between " top hat " C.F. sections and main beams, special backing plates were provided. As will be seen from the selec details in Fig. 14, this plate had to be joggled to all for the thickness differences between connected me bers. Where the connection involved the tape flange of a main member, the joggle, and the sett of the plate on the end of the cold formed memb involved the shops in considerable precision work

Cladding panels were held in position by teak b tens which were pre-drilled to receive $\frac{5}{16}$ in. diame studs welded on the centre line of the face of the C steel members, main beams or stanchions, and pass between the edges of adjacent panels.

Pipes, cable trays, etc., were carried from the roof with $\frac{1}{2}$ in. studs for hangers, and plates for p anchors. Cables and pipes were supported from w by studs or plate brackets, which pass between panels, the batten cover strip being drilled or slot to suit. In most cases the plates were site welded beams and stanchions to ensure accurate location

Phase 3.

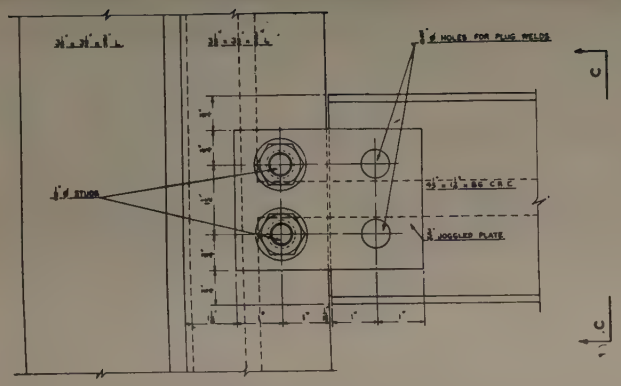
The stage units were now very much smaller and was possible to group them in a more closely pac formation. In addition the stage was made integ with the cell, by building the stage unit into a virt cladding panel.

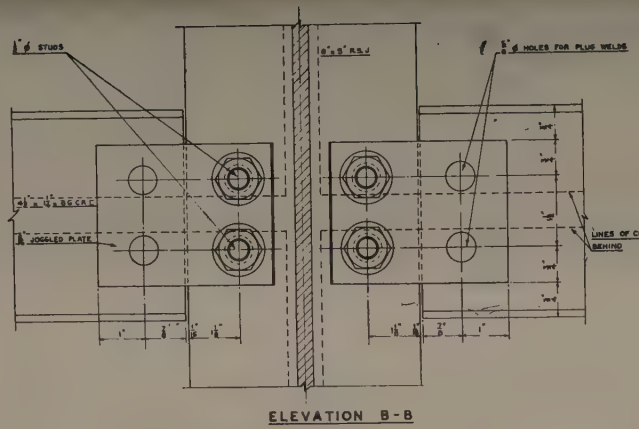
By such an arrangement, it was possible to red the size of cells to approximately 36 ft. 0 in. 15 ft. 0 in. × 13 ft. 0 in. high, grouped in blocks two, forming a block 72 ft. 0 in. × 15 ft. 0 in. Th such blocks, having 12 ft. 0 in. access gangways tween, were grouped together for stability formin structure 72 ft. 0 in. × 81 ft. 0 in. long, the brac system being similar in principle to the earlier ce Once again the arrangement of transformers, swit gear, etc. involved the use of heavy beams, but a c stant beam depth was necessary for the same reas as before. In addition, the plant engineer requi access in two directions for pipes and cables in space between the chequer plate deck and the claddi This led to the use of 12 in. × 6 in. R.S.J.'s for heav loaded beams and 9 in. × 4 in. R.S.J.'s castellated form a beam 12 in. × 4 in., for the remainder. T latter type of construction gave complete freedom running services and cables in a transverse directi

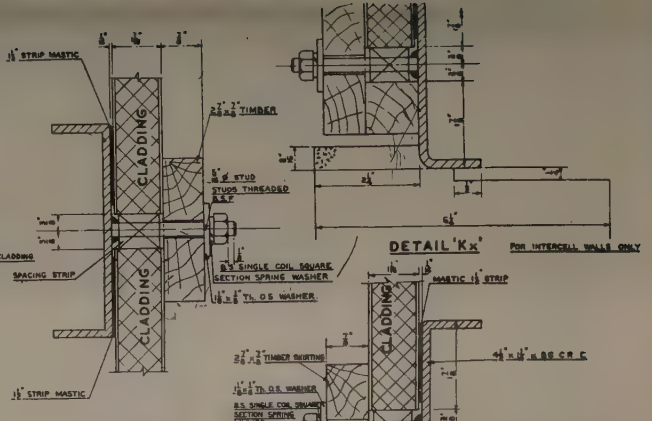
At the product end of the diffusion process, the s of the stage unit becomes extremely small, and economise in space several stages are grouped on ve cal frames, giving a cell size of approximately 7 ft. 6 × 8 ft. 6 in. × 17 ft. 0 in. high. The cells are grou into blocks of 8, each block designed as a stable str ture 30 ft. 0 in. × 17 ft. 0 in. × 17 ft. 0 in. high. erection and maintenance purposes, the whole of cladding and most of the rails in the front face of ea cell has to be removable. The construction is of rolled angle framing posts and cold formed chan and angle rails and roof members. One of the di culties in these cells, was to provide sufficient effect bracing to ensure rigidity, and at the same time le enough space for the forest of pipework inside.

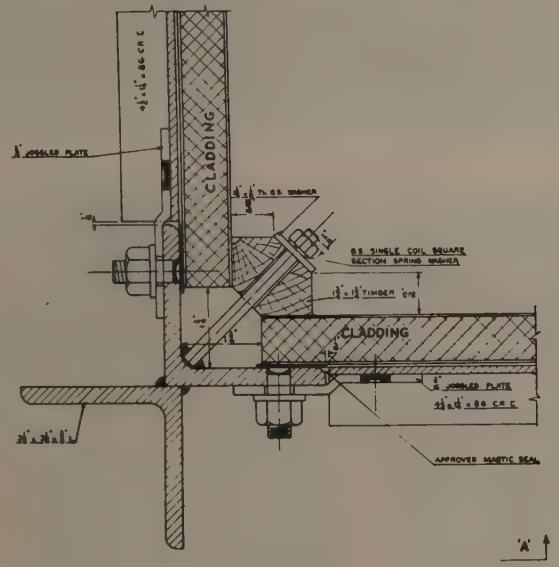
Most of the heavy equipment, consisting of tra formers, frequency changers, etc., was located on ground floor outside of the cell area, the remainder the control gear being located on a chequer plate wo ing deck above at a level of 18 ft. 6 in. supported up a steel frame, and quite independent of the cells.

This working deck was designed as a structure stable units approximately 75 ft. 0 in. × 60 ft. 0 × 18 ft. 6 in. high, independent of the internal s ports of the main building, but with support from side framing beams. It was a requirement of



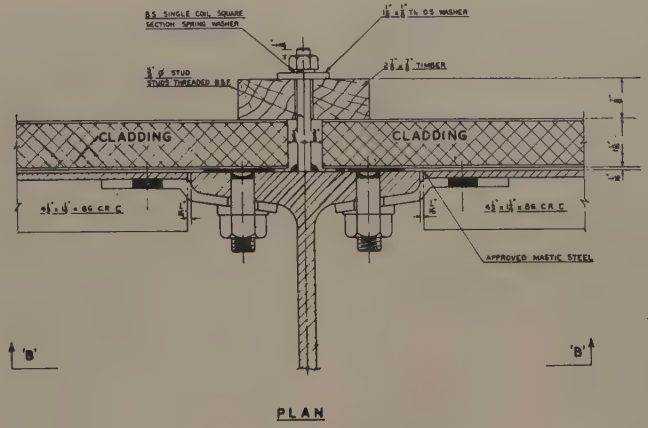






WELDING FOR $\frac{1}{2}$" STUDS MUST NOT PROJECT BEYOND CLADDING FACE

THE CAULKING GAP BETWEEN ANY TWO MEMBERS SHALL NOT BE LESS THAN $\frac{1}{16}$" OR GREATER THAN $\frac{1}{8}$"



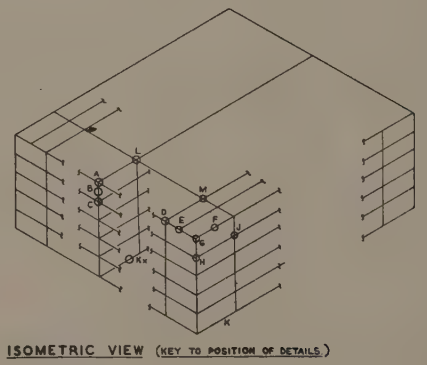


Fig. 14.—Selected details of Cell Cladding for Phase 2

plant engineer that movement of this steelwork should not exceed $\frac{1}{4}$ in at the bus-bar connection between a transformer on the ground floor and switchgear on the working deck. Connections to the side of the building superstructure were detailed as slip joints with copper strip seatings, to ensure that movements in the side of the superstructure were not reflected in the deck, except to the extent of the frictional resistance of the slip joints producing deflection in the vertical bracings. It was estimated that the combined effect of this deflection together with the temperature movement of the deck were well within the $\frac{1}{4}$ in. specified at the bus-bar connection.

Special Techniques Demanded by Special Conditions.

It might be interesting to dwell for a moment on the question of special techniques created by the specified accuracy for the cell structures. With normal steelwork fabricated in structural workshops, there is seldom any difficulty in controlling 'growth' in a building framework. This is probably because the theoretical $\frac{1}{16}$ in. gap usually provided in the connection between beam end cleats and stanchion web or flange face, amply covers for normal over-rollings in steel sections. In the case of the cell steelwork however, not only was the $\frac{1}{16}$ in. minimum clearance between the end of the subsidiary beam and edge of stanchion or main beam a definite requirement, but so also was the $\frac{3}{16}$ in. maximum gap. Moreover it was a requirement which applied at every 4 ft. 0 in. of the height of the stanchions, and for every 4 ft. 0 in. along the length of the main beams.

Special techniques for detailing and fabrication were therefore very important for several reasons :

1. *Rollings.*

A random check on 7 in. × 4 in. R.S.J. sections revealed that the 4 in. wide flange varied up to as much as $4\frac{1}{8}$ in. in width, and the 7 in. depth varied up to $7.\frac{3}{16}$ in. This set the works' drawing office and the shops a real problem if they were to ensure that the cut lengths of beams were to compensate for the unusually wide rolling variations. Secondary members were therefore detailed relative to the *edges* and not to the centres of main steel. By this means in association with systemmatic checks on flange widths, growth in the cell framework was practically eliminated.

2. *Straightness.*

It is surprising how easy it is for main beams and stanchions to arrive on site nominally straight and if they are not quite straight, how easy it is for hot rolled secondary beams to bring them into line. The nominally straight hot rolled sections whether 20 ft. 0 in. or 43 ft. 0 in. long, can be out of true by anything up to $\frac{3}{8}$ in. in their length without this being noticeable, but 8g. C.F. sections will not satisfactorily perform the function of adjustment. The process of checking the straightness of all main members in the shops became essential, and in addition, special emphasis was placed upon methods of loading, unloading, and site stacking, to ensure a minimum degree of distortion as between fabrication and erection.

3. *Cold Formed Sections.*

Again, the use of 8g. cold formed sections called for new techniques in detailing and handling. These may be summarised as follows :

(a) End connections had to be of a type with a minimum projection of end cleats and to provide a maximum degree of nesting during transport for economy in cost, and for stacking on the site.

(b) A completely new approach to the handling of such thin sections in the structural workshops and on the site, in the avoidance of rough treatment.

(c) Systemmatic degreasing of the sections on site immediately prior to painting.

4. *Erection.*

In order to ensure the minimum gap between edges of connected members, a washer pack was temporarily inserted between the members before bolting up, and afterwards removed.

SITE EXPLORATION

Site investigations were made towards the end of 1949. Nine trial pits and five bore holes were sunk revealing from the surface downwards, the following deposits :

(a) Top soil—1 ft. to 1 ft. 6 in. thick.
(b) Boulder clay—Nil to 18 ft. thick.
(c) Sandy clay—Nil to 13 ft. thick.
(d) Compact red sand—4 ft. to 25 ft. thick.
(e) Coarse red sandstone occurring at levels 25 ft. from surface to over 60 ft. from the surface.

It was found that the boulder clay deposits were haphazard and irregular in depth. They covered the whole of the factory site except at the north west corner, where sand appeared at the surface.

The information thus obtained, established that the Process Area would be built on clay, but which would be variable in thickness.

On the boulder clays and sandy clays, triaxial compression tests, classification tests (Atterberg limits), consolidation tests, natural moisture content determinations and shrinkage tests were carried out. These latter tests showed a natural moisture content of 14.7 per cent. and a shrinkage limit of 12.1 per cent. This was important in the process area and will be referred to later.

As a result of the laboratory tests, it was decided to impose $2\frac{1}{2}$ tons per sq. ft. mean pressure from main foundations on the boulder clay with an edge pressure not exceeding 3 tons per sq. ft. Both the sandy clay and the sand below it were found to be suitable for similar working pressures.

FOUNDATIONS

Phases 1 *and* 2.

General.

The details for these two phases are so similar that for convenience they may be dealt with together.

Main Building.

The presence of the cooling water return mains along the corridors prevented the introduction of a cill beam to fix the portal feet, and the proximity of the cell slabs in the bays severely restricted the width of the portal foundation blocks, for which reasons it will be recalled, it was not possible to obtain an measurable degree of fixity in the portal feet. Nevertheless in addition to the vertical loads and horizontal wind shears, the foundation blocks were designed to accommodate a bending moment equivalent to the value of the two fixing bolts for the portal feet.

The foundations for the wind bay along the corridors ere designed as R.C. strip footings to resist an uplift f 30 tons due to wind.
ells.

Conditions laid down.

By far the most important foundation problem was hat for the cells.

The conditions inside the cells were as follows :

1. The working temperature would be maintained at 140°F.
2. The relative humidity would be maintained at less than 2 per cent.
3. Relative settlement in the floor slab in any one group of cells had to be avoided.
4. Dusting conditions inside the cells had to be avoided.
5. Stage unit feet were to sit on steel pads cast in the floor slab. It was stipulated that these feet were to be set to an accuracy in position and level to 0.005 in. to ensure interchangeability of stage units with their associated pipework along the process line.
6. No " stepping " would be allowed between stage feet and grano floor.

Let us consider the effect of items 1 to 3 upon the clay below the slab. Heat within the cells would be transferred to the concrete slab which would then be subject to a temperature rise and because of the extremely dry internal conditions would readily lose moisture to the air inside the cell, this moisture being subsequently carried away by the air driers, which are located on the top of the cells. Similar heat and dryness conditions would eventually develop in the clay which would tend to dry out, and therefore to shrink, in the zones immediately below the cells.

In a structure so large, climatic conditions, and in particular storm water could have little or no effect upon the clay under cells, i.e. in the replacement of moisture which had been drawn into the cells, particularly if the cells happen to be located centrally in the process building. In other words the clay would slowly dry out. The relationship between the natural moisture content of the clay and its shrinkage limit was therefore quite important.

Design approach.

From the soil mechanics tests the average natural moisture content of the clay was found to be 14.7 per cent. and its shrinkage limit 12.1 per cent. This meant that complete drying out would involve a volumetric reduction of only 2.6 per cent., with a corresponding linear shrinkage of little more than 1 per cent. It was clear then, that long term shrinkage problems from this cause did not exist so long as the natural moisture content of the clay were maintained during construction.

The size of a cell group or block is in the region of 108 ft. × 86 ft., and laid side by side, the cells occupy the whole of the space in the bays between the portals. This is a large area for the control of thermal movement in concrete, but the importance of maintaining dimensional relationships between stages within the cells prohibited the introduction of expansion joints in the floor within this area.

It was obvious, of course, that freedom of movement for the cell slab to expand and contract were of prime importance. A reinforced concrete slab approximately 110 ft. long by 90 ft. wide, flat on the underside was accordingly adopted as the best form of foundation, having an expansion joint of the pre-moulded filler type around its perimeter, and a form of sliding membrane on the underside.

STRUCTURAL DESIGN

Cold worked steel was used as reinforcement to the slab, two layers being provided beneath the stanchions and designed to act with the slab as continuous footings. In the remaining area of the slab, two layers of a smaller cross sectional area were employed to deal with the drag due to thermal contraction of the slab. The nature of the stresses in the reinforcement was likely to be twofold :

(a) Bending stresses of a permanent nature induced by stanchion loads and for which design stresses were limited to 27,000 per sq. in.

(b) Tension stresses due to frictional resistance to thermal contraction induced very infrequently during a period in which maintenance work was taking place in a cell. Design stresses were increased to a maximum of 30,000 per sq. in.

CONSTRUCTION WORK ON SITE

The problem of how best to maintain the natural moisture content of the clay during construction was given considerable thought and the following procedure was finally laid down for the construction work in the process area.

1. An adequate and carefully planned drainage system was to be provided at the outset to deal with storm water. To accomplish this, the laying of the process water return mains was completed corridor by corridor, ahead of the work in the cell area.
2. The formation in the cell area was to be graded to a slight fall into these drains.
3. The clay was to be exposed for as short a period as possible. This was successfully achieved by systematically sealing with Prefabricated Bituminous Sheeting (P.B.S.) the exposed formation close behind the grader. To facilitate this, the P.B.S. was laid over the whole of the area and turned into the drain excavation, the concrete for the drain surround being arranged to form a natural channel for carrying storm water into openings left in the mains already laid.
4. The P.B.S. was to be subsequently cut diagonally and folded back at the main building foundations, and afterwards turned down into the excavations before concreting.

By such means, and by limiting the pressure on the clay below the cell slab to a maximum of $\frac{3}{4}$ ton per sq. ft., the likelihood of differential settlement and/or cracking in the slab appeared to be remote. The P.B.S. below the slab was regarded as a reasonably satisfactory sliding membrane for thermal movement.

The slab was cast in alternate panels approximately 15 ft. 0 in. wide × 30 ft. 0 in. long with a minimum period of 72 hours between adjacent panels. The mesh was ordered and supplied in panel sizes to suit the casting programme to ensure speed of placing and correct lapping.

Pads for stage unit feet were set in pockets 6 in. deep and the floor grouted up to give a smooth and clean joint. The accuracy called for in setting the pads involved the use of special datum plates and setting out plates. These were cast in the floor and levelled to an accuracy of ± 0.002 in. The jigs for setting the stage unit pads were located and levelled from these.

The specified accuracy in the finished level of the concrete slab in a cell 55 ft. × 45 ft. was that the grano surface must lie wholly between two parallel planes ¼ in. apart, and not more than ¼ in. above or below datum. This called for considerable care in setting and levelling and constant checking of the screeding boards, particularly when an added complication of laying the 1½ in. granolithic finish integral with the slab was included in the operation. A disc float was used for this last operation, and the surface produced was good enough to dispense with the need for any proprietary surface hardener.

Concrete.

In an attempt to save cement in the concrete work without sacrificing quality, a " guaranteed strength " specification was laid down. It called for weigh batching of 1½ in. down aggregates, and a stringent control of the water cement ratio.

The permissible range of aggregate—cement proportions laid down in the specification was from 8.5-1, to 7.1-1 by weight, to give a guaranteed strength of 3,000 lbs. per sq. in. at 28 days. Trial mixes were expected to give an average of about 4,000 lbs. per sq. in., and in order to achieve this, a considerable effort was put into the early trials at the site before a mix of 7.1-1 was finally adopted.

Internal and pan vibrators were used to compact the concrete.

Phase 3.

In these cells, growing progressively smaller in size, an additional complication arose. The stage units, as already mentioned, were virtually made integral with the cell walls. This called for extreme accuracy in the concrete finished levels, to ensure an absence of ' lipping,' ' ramping,' or ' sinking,' with the stage unit feet, which had been set to levels ± 0.002 in.

At the ' Product ' end of the process, the plant engineer was so anxious to maintain a dry atmosphere within the cells that asphalt tanking was laid below the slab and vertically around its perimeter up to finished ground floor level, to act as a barrier to moisture in the clay below, or in the surrounding floor slabs. Whether in fact the asphalt acts as a completely effective moisture barrier under the exceptionally severe working conditions imposed upon it, time will tell.

Apart from variations of loading and details of stage feet pads, which were much more numerous, the general design principles for the cell slabs in Phases 1 and 2 were adopted for stage 3.

ASSOCIATED SERVICES BUILDINGS

General.

As their name suggests this heading covers the buildings in which everything required for the servicing of the process is carried out from the point where the original gas is fed into the line, to the handling of the final enriched product and the unwanted wastes. They are all attached to the process building.

Roof Design.

These buildings are of 40 ft. 0 in. and 80 ft. 0 in spans 30 ft. 0 in. high to eaves, the former having three ton and five ton overhead electric travelling gantries bracketed from stanchions, and the latter having one and half ton and three ton blocks slung from the roof. The roof of the 80 ft. 0 in. span buildings which was designed to link with the roof of twin 40 ft. 0 in. span buildings previously completed, is perhaps interesting and is reproduced in Fig. 15.

It will be seen that the design is in tubes except for the main tie which is in hot rolled steel.

The basic price of tubes was more than twice that of hot rolled steel sections so that even with a saving in weight of say 33⅓ per cent., the cost of raw material is still greater. Not only did the introduction of the hot rolled tee section as a main tie save initial costs but the simplified end preparation of web members connected to it helped to reduce fabrication costs. The design was an attempt to get the best of two worlds. The remainder of the building followed a fairly normal pattern.

Process Structures.

Within these buildings and quite independent of the main frame are erected an assortment of plant ' Process Structures ' for the Feed, Product and Waste. These structures carried all items of plant, services, cables, electrical equipment, maintenance platforms, access gangways and the like, and careful consideration had to be given to the stability of each group of structures, as the plant engineer reserved the right to remove or modify any group at a later date to accommodate possible later developments.

The tonnage of structural steel in the process structures within three of these buildings amounted to over 500 tons.

Hot Boxes.

Quite apart from, but associated with the plant process structures, a considerable number of ' hot boxes ' were provided. Within these hot boxes were a concentration of pipes and valves, which because of their nature were extremely difficult to lag, and as their names suggest, the boxes were a means of providing controlled conditions around the pipes and valves, making individual lagging unnecessary.

The sizes of the hot boxes varied very considerably from 7 ft. 0 in. high × 2 ft. 6 in. wide × 40 ft. 0 in. long, to approximately 25 ft. 0 in. high × 8 ft. 0 in. wide × 50 ft. 0 in. long, but they all had in common a requirement which prevented the use of internal diagonal bracings. In addition because of their association with interchangeable plant, etc., an unusual degree of accuracy was again laid down for finished sizes of the boxes, in the alignment of ' operating faces,' together with limitations similar to those imposed on the cell steelwork, to conform with similar standards of insulation.

The hot box bents were designed and fabricated as portals and the usual erection methods of guying the steel into alignment before grouting was therefore not acceptable, as removal of the guys after grouting would be followed by a springing out of line, equivalent to side sway after removal of a prop. The methods used for adjustment were by shimming up from floor level until the required line along the top of the box had been achieved within the accepted tolerances.

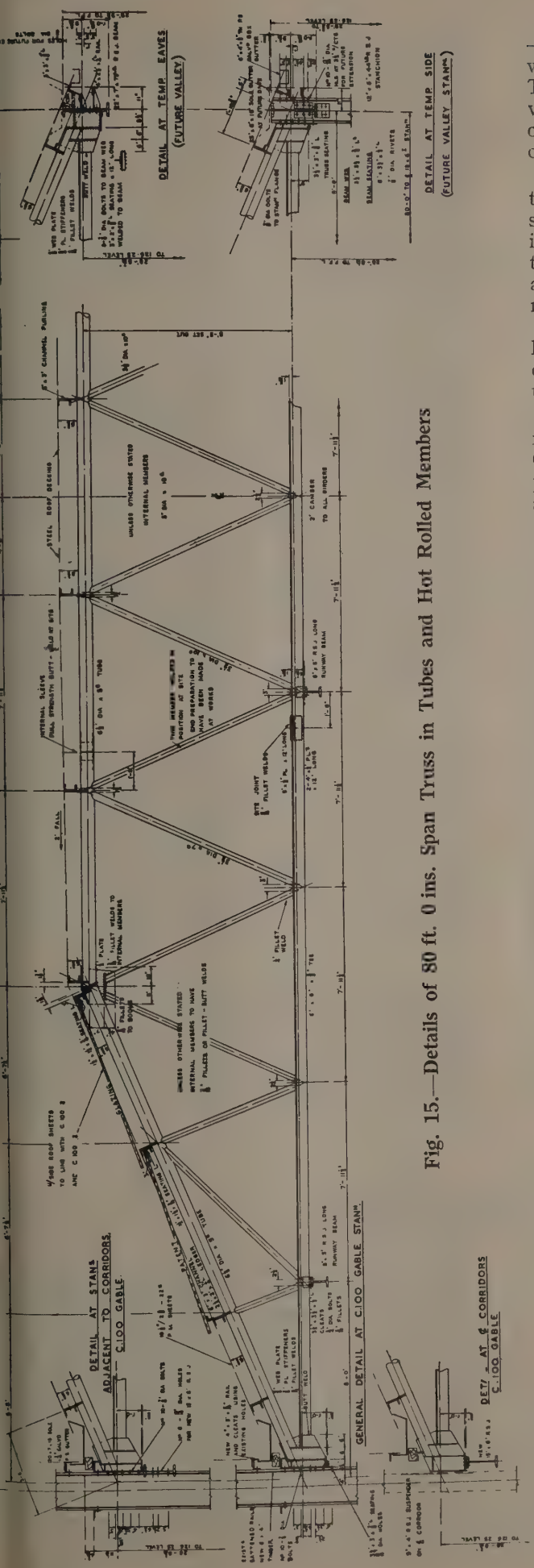

DETAIL AT TEMP. EAVES
(FUTURE VALLEY)
DETAIL AT TEMP. SIDE
(FUTURE VALLEY STANN.)
GENERAL DETAIL AT C.100 GABLE STANN.
DETAIL AT STANN. ADJACENT TO CORRIDORS C.100 GABLE
DETT. AT ℄ CORRIDORS C.100 GABLE

Fig. 15.—Details of 80 ft. 0 ins. Span Truss in Tubes and Hot Rolled Members

The hot boxes—like the cells in the process building—were clad with 1in. thick aluminium faced cork through which, at intervals, valve spindles and pipes penetrated. The boxes were usually sub-divided into compartments varying in size, to permit maintenance work in one compartment to be carried out without affecting process work in others.

It was therefore a requirement of the plant designer that for maintenance purposes most of the panels should be easily removable from the outside. This involved a fundamental change in the fixing details of the panels, cover strips which retained in position two adjacent panels being no longer permissible. As a result, the panels which varied in size up to 8 ft. 0 in. × 4 ft. 0 in. were provided with timber edging strips having holes for studs, and were fixed to the framework of the hot boxes by $\frac{5}{16}$ in. diameter studs which passed through these holes.

Stud welding to the steel frame was by ' panel jig ' method to ensure a satisfactory fit for the cladding. Centre popping was carried out through holes in the panel jigs after which the jigs were removed and the studs welded into position.

BUILDINGS VERSUS PLANT

The relationship between costs of building and civil engineering work on the one hand and those of the plant on the other were known to be of the order of approximately 1 to 10. The electrical work alone cost $2\frac{1}{2}$ times the building and civil engineering work. The penalty for delays in the installation of the plant, and therefore to the start up of production was a very serious one.

This consideration largely dictated the pace and the order in which the various buildings or parts of buildings were erected and handed over. It also emphasised the importance of designing and if necessary of adjusting the building structures to suit the needs of the plant (as yet unborn in many cases, especially in the Services Buildings), rather than that of designing the plant to suit the structures. Nevertheless as the work proceeded there was a considerable amount of give and take, as between the requirements of the plant, and the limitations of the building umbrella, a fine line being drawn between what was desirable and what was essential.

In addition, sweeping decisions often had to be made by the plant designer on building sizes, cranage requirements and the like, amounting sometimes, due at that time to lack of knowledge of plant types and lay-outs, just to intelligent " guess work," in order to keep structural design and construction moving according to a pre-arranged programme.

As already described some of the conditions laid down by the plant designer called for unusual and unexpected structural refinements—the need for extreme accuracy in fabrication and erection of steel work, novel detailing techniques, special consideration of thermal movements, the effect of relative settlement in floor slabs, the laying of granolithic floors to precise levels, etc.

ERECTION OF THE PROCESS BUILDING

Portals.

It was intended originally to rivet the portals completely on the ground and to lift the assembly into position (total weight $7\frac{1}{2}$ tons) by means of an 8 ton 60 ft. jib mobile crane. The longitudinal ties would then be lifted and connected to the portals with service bolts for subsequent riveting. It was found, however,

that as riveting up aloft was necessary for the longitudinal beam connections, time could be saved by lifting the portal legs separately and fixing the cross members and the longitudinal members in situ with service bolts, and making the lateral and longitudinal riveted connections after plumbing and lining had been carried out. This in fact became the accepted procedure on the site for all subsequent corridor steelwork.

Trusses.

The trusses were fabricated and delivered in four pieces, riveted up on the ground, and in the first bay, lifted into position by two 60 ft. 0 in. jib mobile cranes. Subsequently a five ton, 100 ft. 0 in. jib Scotch derrick replaced one of the mobile cranes.

The trusses were followed in order by the longitudinal girders, intermediate rafters, purlins, bracings, longitudinal ties, cable brackets, services supports and walkways, and no sooner had the first unit been lined, plumbed and riveted, than roof and side cladding commenced.

Cells.

Two months after the first portal was lifted, erection of the cell framework commenced, and thereafter it kept pace with the erection of the main building. As soon as the cells were lined and plumbed, block by block, the erection of chequer plate decking commenced on the roof of the cells. Thus two months after lifting the first corridor portal, erection of building and cells were proceeding concurrently.

Generally.

Erection of the process building started mid-October, 1950, and within three months, steelwork was being erected on the site at the rate of 950 tons per month until well into the summer of 1951 when the first phase was nearing completion. As Phases 2 and 3 followed Phase 1, the tempo was maintained apart from a short hiatus between the phases. In the period mid-October 1950, to mid-May 1954, approximately 20,000 tons of light structural steelwork, including approximately 800 tons of tubular steel construction and 350 tons of cold formed steel sections were erected.

CONCLUSION

It has been possible to dwell upon a few of the more interesting aspects of this project. It has not been possible to present the atmosphere which prevailed during the period of design and construction.

A construction programme was laid down at the end of 1949 which aimed at commencing factory production at a given date and this was a goal which had to be achieved in spite of any difficulties which might arise during the course of design or construction.

Many aspects of the plant were quite unusual and even revolutionary. Its design had only just commenced and details were far from finalised. The unknown factors were therefore many and varied in the early days of structural design which had to proceed hand in hand with or sometimes before the design of the plant, and a good deal of intelligent guesswork was resorted to on those aspects of the buildings which were tied up with such plant. In fact as work proceeded, it was necessary to pick up information progressively and add to or modify drawings which had already been produced and issued to the construction engineers. It was sometimes necessary to order steelwork before a final design analysis had been made in order to ensure that erection dates would not be jeopardised.

As each successive alteration was received from the plant designer, every effort was made to pick it up before fabricated steel left the shops, and this became an accepted practice, provided the resultant delay did not interfere with the erection programme. Often the steelwork was actually passing through the shops these modifications were usually made on the site.

With alterations due to finalised plant design coming through at regular intervals, delays in the issue of modified structural detail drawings threatened at times to jeopardise the site programme. Such circumstances called for special measures and a 'cut-off line' date was usually arranged and agreed with the constructional workshops, who would fabricate to drawings in their possession *on that date*, further modifications being picked up on the site. The dividing line between deciding upon shop alterations and site alterations was often a very fine one, but with good will and co-operation on both sides, the very tight target dates set were always met and often beaten.

ACKNOWLEDGMENTS

The work was carried out under Sir Charles M... Director General of Works, Ministry of Works, ... Sir Christopher Hinton, Managing Director of ... Industrial Group of the Department of Atomic Ener... from whom permission to publish this paper has b... received.

Book Reviews

Mechanical Engineering Workshop Practice, by E. A. Peatfield, A.M.I.Struct.E., A.M.I.Mech.E. (English Universities Press Ltd., 1952.) 203 pp. 7½ in. × 4½ in. 6/-.

This book is a new one and shows clearly how the author appreciates the difficulties of students. The illustrations are beautifully done mostly in isometric and the text goes steadily but clearly through the various processes. Drawings, the method of reading them, pattern making, are nice leads in to a very good chapter on foundry work. It is good to see that blacksmiths work and forgery get their due pla... Since this book is one of the "teach yourself" ser... the author has quite rightly assumed little previ... knowledge from the reader and so made the expla... tion clear on points which are often missed as being ... simple. This is particularly true of the chapter ... machine shop work, which uses 92 pages. A very l... chapter but full of good stuff. Welding, fitting ... benching and draughtsmanship complete a book wh... is very well written and illustrated. It will be us... to the student for whom it is written.

N. T...

(*continued on page 296*)

he Development of the Elastic Theory of Continuous Frames*

Discussion on the Paper by Dr. E. H. Bateman, M.A., B.Sc., M.I.C.E., A.M.I.Mech.E., A.F.R.Ae.S. (Member of Council)

TRODUCING the Author of the paper, the PRESIDENT said Dr. Bateman was elected to Associate Member- o of the Institution in 1932 and transferred to full mbership in 1949, and at the present time he was the staff of the Royal Aircraft Establishment at nborough.

Dr. Bateman was Hon. Secretary of the Midland nties Branch of the Institution from 1934 to 6 and was a member of the Branch Committee until 9; he had been a Member of Council since 1952, ving on the Literature Committee, the Science and search Committee, the Education and Examinations mmittee, and the Ad Hoc Committee on Aluminium oys.

Concluding his remarks, the President said Dr. teman had already given a notable paper to the titution, in 1936, on the subject of "The Stress alysis of Continuous Frames," in which he compared various methods of treating the subject, and for ich he was awarded the Institution's Bronze Medal. had now written another paper which was a most portant contribution to the history of the subject, d again presented a valuable comparison of methods.

Dr. E. H. BATEMAN said that when he had been ked, three years previously, to write a paper re- wing the various methods of solving indeterminate uctures he had thought it might seem to be flogging dead horse to refer once more to elastic theories design. However, he had eventually undertaken e task, because he felt that the plastic theory should t be applied indiscriminately to every problem in e design of frames.

Those who had read Lord Cohen's Report[81] on e Inquiry into the loss of the Comet aircraft would preciate that it was not enough to prove a structure estimating a load and doubling it and then applying to that structure, if the structure was one on which e loads might be repeated. Fatigue was not merely matter of a small alternating load within the elastic nit repeated many millions of times; there was the gh level fatigue which would bring one to grief in few thousand cycles, so that for structures in which e live loads might predominate there was still a od case for considering elastic methods of analysis.

Pointing out a misprint which had occurred in the paper, Dr. Bateman said that on page 76, column 1, the last paragraph, in which he referred to Fig. 2, there appeared the words: "If the required quantities are deformations, there are 3 joint rotations," but that should read "13 joint rotations."

Dr. Bateman then presented his paper, which was illustrated by slides.

The PRESIDENT, proposing a vote of thanks to the Author, said the paper was not over mathematical—in fact he had found it a most interesting historical account and having himself delved into that history some years ago he was naturally very interested in its treatment by Dr. Bateman. One thing he had been particularly pleased to find was the emphasis which Dr. Bateman had placed on the work of Otto Mohr because Mohr was one of those people who seemed to have done many things for which other people got much of the credit. Much of his original work was published in an obscure journal in Hanover where only a few people ever saw it. The President had once tried to trace one of Mohr's key papers and had been unable to find it in London in any of the special libraries. Mohr started his career as a railway engineer, and in the 1850's he was designing iron railway bridges in pin-jointed truss construction. They must have been among the earliest railway bridges of that type outside the U.S.A. where they were mostly of wood. Our own Board of Trade was discouraging engineers in this country from using trussed iron girder bridges then and for a long period afterwards.

At 33 Mohr became professor of engineering mechanics at Stuttgart. He held that post for some years and developed many of the methods of graphic statics which became so useful in the latter part of the century. It seemed also that he was responsible for the first use of the influence line although the credit for this was usually given to Winkler, and for the influence line for deflection, the credit for which was generally given to Müller-Breslau. It was quite time that someone in a position to study the subject from the original German sources gave us a technical biography of Mohr, because he had evidently done a great deal more work than was usually credited to him, including the application to frames of the slope deflection method, for which Maney claimed the credit as late as 1916 whereas the date now given for Mohr was 1892. The method had, of course, been used in 1879 by Manderla in studying the secondary stresses in trusses.

*Read before the Institution of Structural Engineers at 11, pper Belgrave Street, London, S.W.I., on the 24th March, 1955. r. S. B. Hamilton, Ph.D., M.Sc.(Eng.), M.I.C.E., M.I.Struct.E. resident) in the Chair. Published in "The Structural Engi- er," Vol. XXXIII, No. 3, pp. 73-83, March, 1955.

Dr. Bateman had found one misprint and although he was sorry to point out another the President said that at the top right hand side of page 77, equation (3a) the subscript A's and B's had been transposed.

Referring to the use of electronic computers : if one were to believe the popular press one might imagine the engineer of the future sitting down in his arm chair, smoking a fat cigar, while his machine solved all his problems for him and he only had to pass the information on to a junior on the drawing board ; but Dr. Bateman had given a timely reminder that a great deal of mathematical skill was required on the part of the man who had to prepare the material to feed to the machine. Electronic computing would save an enormous amount of routine labour, which after all never was real thinking, but it would do nothing to reduce the amount of actual brain work which had to be done by the man designing the job.

The vote of thanks was accorded with acclamation and was responded to briefly by the Author.

Discussion

Professor A. L. L. BAKER (Honorary Treasurer) said there were few men in the country who could have attempted a review such as Dr. Bateman had carried out in his paper and he was to be complimented and congratulated on such a broad survey and for all the astute comments which he had made. He heartily agreed with Dr. Bateman on his comments about the need for variety in method according to the type of structure, according to whether the calculations were preliminary ones, according to whether one was analysing or designing, and so forth. If there was one comment which might have a slightly critical aspect it was in the need for an attempt to assess and value those alternative methods which had been referred to a little more closely, and he personally would suggest that more emphasis should be given to the value of the work of Müller-Breslau. On the Continent, and working with Continental engineers, he had found that the Müller-Breslau general elastic equations which could be applied to any type of statically indeterminate structure had been in common use for about the last 50 years. It seemed to be regarded as the classical theory which students were taught and designers used in dealing with all kinds of statically indeterminate problems, simply because the equations were always the same and the solutions therefore were always the same. Moreover, with the Müller-Breslau equations one could derive them directly from simple fundamentals. (*Illustrating on board*). The first fundamental step in all statically indeterminate theories—was to establish the simple fact that the change of slope along the member was the integration of the bending moment acting on the member divided by the EI value—that gave the engineer a visual impression of the deformation in any member of a framework, and one could go straight from that fundamental to the general equations of Müller-Breslau. There was no question of computing strain energy and adding it up and differentiating and possibly worrying the mind with why the total should be a minimum. A simple rule took care of signs.

One could see the load—stress—deformation relation clearly : one simply took a complex framework, perhaps many hundreds of times statically indeterminate, and put in sufficient hinges or cuts to break the whole framework up into a number of statically determinate systems so that one had t a set of statically determinate pieces of framew which were all joined at the hinges or cuts. (*monstrating on board*). So straight from first princi one got a standard set of equations for the ela condition, based on the condition that there was break in the continuity at the section which l either been cut or had a hinge inserted, an unkno bending moment and/or force being inserted to rest the original continuity. If the framework was times statically indeterminate one obtained n eq tions. Those equations were of the same form wh ever the framework, so once one had that set of eq tions and had solved them the general solutions w available for other cases. In these days with help of the digital computer one could go up to h dreds of unknowns. Müller-Breslau's method been used for thirty or forty years by the Continen workers and it had been very slow in coming to country. Perhaps Dr. Bateman could say whet the little book which was published by Dr. Gartn in 1938 was the first time those equations of Mül Breslau had appeared in an English text book ?

Professor Baker said if he might be forgiven bringing in a personal reference, the story of t little book might be of some interest. It was in one of the good things which came out of the Hi regime and the economic depression in this count in 1936 Dr. Gartner was an eminent engineer work in Germany but he went to South Africa to get a from the Nazi regime and at the same time he (P fessor Baker) had gone there to get away from economic depression. Dr. Gartner had taught the Müller-Breslau methods and it had occurred to him that there was nothing like his presentation in the English text books and he had therefore offe to help Dr. Gartner if he would write and exp them in English. The book was subsequently p lished, and he believed that was the first time Müller-Breslau's notation and equations had b presented in a text book in this country. Now course, those equations were being taught at Impe College and, he believed, at the Hammersmith Sc of Building and Arts and Crafts.

Another interesting thing about those equat was that they applied and fitted in very well with plastic theory. They were simply transformed. unknowns of all the hinged points became kno and in each equation one had, instead of n unkno only one unknown which was the angle of dis tinuity which occurred at the plastic hinge. (*I trating on board*).

Professor Baker said he agreed with Dr. Bate that the plastic theory needed to be used with cretion and with caution. In most framework was probably necessary to do an approximate c of the distribution of bending moments under wor load to ensure that, through fatigue or excess of st which might occur under a working load in a framew one did not get weakening or serious cracks in reinforced concrete. It was usually necessary only to do the plastic theory calculation but als approximate elastic theory calculation.

There was a tremendous difference between approximate elastic theory calculation and a r rigorous one. The approximate one could be quite quickly with the same set of equations and same set of hinge points, and by selecting the co hinge points for the plastic theory one automati

oduced equations which were not ill conditioned, nich meant that a very large number of equations uld quickly be solved approximately if the right ethods of attack were used. In that connection might be appropriate to draw attention to a method scribed by Dr. Yu in a discussion on a recent paper nich he (Professor Baker) had given to the Institun of Civil Engineers[83], because in his opinion Yu's ork was quite important: he was a student at nperial College and he had investigated the plastic nge theories and methods of using those equations order to try and obtain improvements. Yu had und that if he set out his equations and then followed particular method of trial and adjustment, which as really a relaxation method, i.e. if he adjusted e hinges in the beams first, coming down through e building so that his resultant deformation in ch equation was zero, thereby showing there was correct elastic solution, and if he then started at e top again for the inside column hinges, and then he finally took the outside hinges starting at the p—by that simple method with about two cycles the operation he obtained approximately zero value for all his resultant rotations or angles of discontinuity very quickly. Yu was a student from Hong Kong and he was proud of the fact that he could perform his adjustment calculations quickly with a Chinese abacus, and he rather laughed at the electronic calculating machine for dealing with building frames, and that was probably the right spirit. Professor Baker said he would not for one moment deprecate using the electronic machine where it really was of advantage but to his mind the simpler the calculating tool the better the method which was being employed because the engineer could see so clearly the relation between moment distribution and the geometry of a frame and the varying stiffnesses in one member and another—in fact he could design as well as analyse at the same time.

Dr. R. H. Wood (Associate-Member) said that he had just been at the Royal Institute of British Architects where a paper was given by Professor Fisher Cassie and Mr. Cooper and the predominant mood there was plastic design. Now Dr. Bateman had reviewed the development of elastic theory, and it would appear

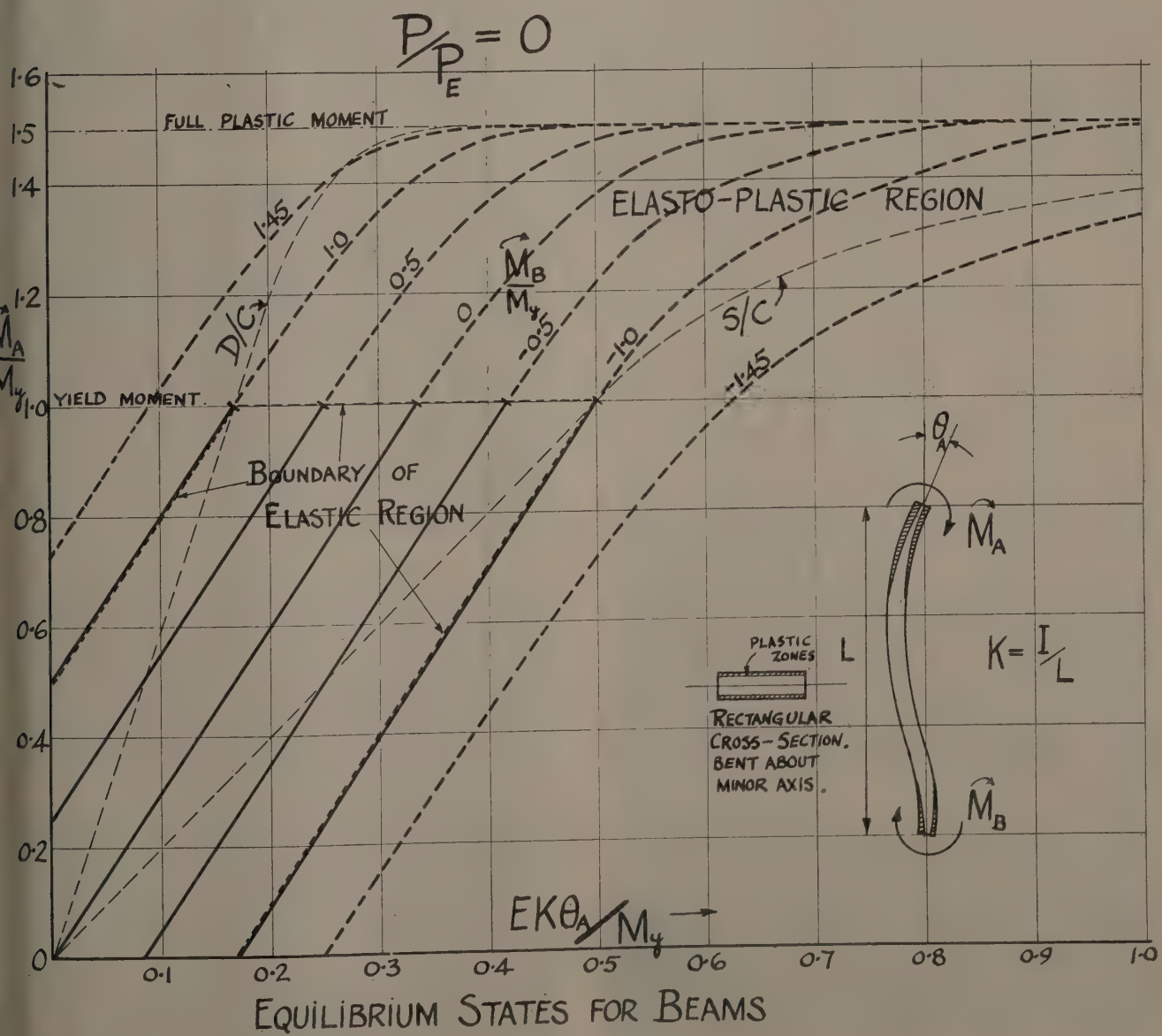


Fig. 1.

to be somewhat confusing as to what line engineers should take, and he hoped to contribute towards putting the subject into its proper perspective.

The paper was undoubtedly a fine review of elastic theory. He had not known, for instance, that Otto Mohr was responsible for slope-deflection equations; like others, he had formerly attributed it to Wilson and Maney. He had also liked the particular form of the strain energy equations. However, partial differentiation of equation (8) gives equation (4a), which was a slope deflection equation. He could not for the moment see where the special claim arose in strain energy methods, and perhaps the Author could shed a little more light on that point.

Dr. Bateman had written: "In view of recent progress in the plastic theory of design it may well be asked whether there is in fact any need to devote further attention to the elastic theory." In part, that question had been answered by the present paper, but if one turned on a few pages in that same issue of the Journal one would find the real answer as to why it was impossible to depend solely on plastic design, and that was in a fine paper by Mr. Merchant on "Critical Loads of Tall Building Frames."[84]

In his introduction the Author had quite deliberately decided to leave out the effects of direct load, but as regards the historical side, and in view of the Author's propensity with matrices, it was rather surprising that Berry and his work had not been mentioned, particularly as it was used by the Steel Structures Research Committee. It did not seem to be very widely known that the slope deflection equations transformed very neatly if one used functions X and Y (themselves of Berry functions), as he had himself outlined in a previous paper to the Institution. Thus:

$$M_A = 2EK\left[2\theta_A + \theta_B - 3R\right]$$

becomes very simply

$$M_A = 2EK\left[2\theta_A.Y + \theta_B.X - R(2Y + X)\right]$$

Dr. Wood then showed slides depicting the most recent results of elasto-plastic analysis of stanchions obtained on the Building Research Station differential analyser. These showed elasto-plastic states expressed as moment-rotation (M/θ) relationships, as extensions of the slope deflection equations (Figs. 1, 2, 3). The effects of the onset of plasticity were instantly clear. The answer to the question "Why should we bother about elasticity?" seemed to be fairly obvious: if one combined the comments of Dr. Bateman, the contribution of Mr. Merchant and the properties shown in his own slides then possibly the question should be put the other way round, namely, "Dare we abandon elasticity altogether?"

Mr. A. Goldstein (Associate-Member) said that it was a great pity that we did not read more about the experiences of our engineering ancestors. The teaching of the historical development of elasticity should be included in the curricula of most universities and colleges. Referring to Page 75 of the paper, where Dr. Bateman gave twenty-five methods—and in his introduction referred to them at times as theories—he thought that when examined basically it was overdoing things a little to call these techniques distinct methods or theories. There would appear

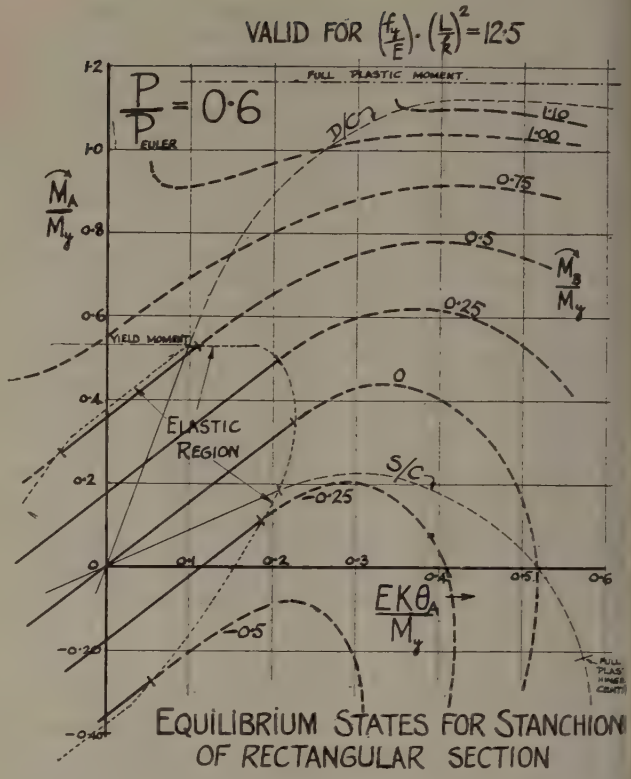


Fig. 2.

to be basically two methods of analysing structures there were either the direct, analytically precise solutions or the indirect—analytically not precise but equally accurate—solutions such as relaxation methods etc. If one agreed with this fundamental distinction then admittedly there were many ways of applying the direct methods. Often when faced with a problem one put down in general terms the various equations governing the particular problem and then the arrangement for computation is a matter of personal predilection, and, depending on the arrangement, one would end up with one of the detailed techniques given on Page 75. In Mr. Goldstein's opinion, these techniques could hardly be said to differ fundamentally.

The technique of flexibility coefficients which Professor Baker had also mentioned in his contribution was indeed a most powerful and often overlooked tool for the designer. It is particularly elegant when combined with matrix methods and terminology. Apart from the saving in computation time engendered thereby, a considerable advantage is the fact that one can pursue theoretical arguments using matrix notation with much less effort than when the detailed classical terminology is used.

Mr. Goldstein noted that Dr. Bateman had mentioned the digital computor, and thought that analogue computers might also merit some reference. Referring to his earlier distinction of direct and indirect solutions, it could be said that the digital computer was the mechanical counterpart of the indirect solution in moment distribution etc., whereas the analogue computer was the mechanical counterpart of the direct solution. The two types of computer could, of course, be used in conjunction with each other. He would welcome Dr. Bateman's views on these points.

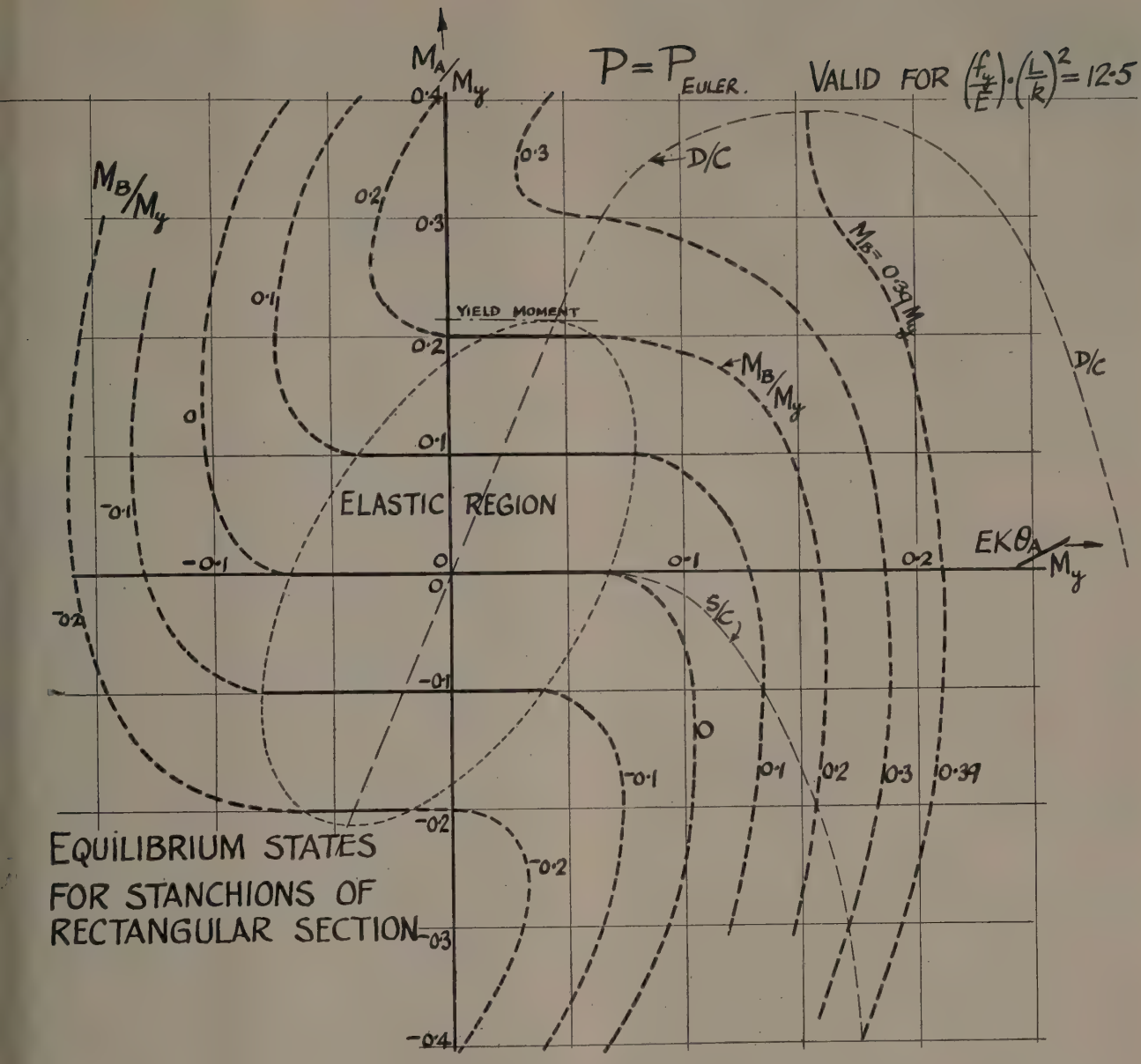


Fig. 3.

Mr. R. J. Wilkins (Member) said on page 75 of his paper Dr. Bateman had made a statement which had appealed to him: "No one approaching the subject with the mind of a novice or a layman could fail to be bewildered by the abundance and apparent variety of the methods offered or advocated by the many writers who had ventured into this field," and there followed a list of 25 proposed methods, so he had ticked off the number he could expect one of his students to have heard of although he would not say they would be completely familiar with them. The list would be Beam Theory in the first year, keeping quiet about all others; then Macaulay's method and, later, the Theorem of Three Moments; Virtual Work, Strain Energy, Arch Theory, Moment-Area, Characteristic Points, Slope-Deflection, Relaxation and, finally, Column—Analogy. He had taught at Higher National, Professional Institution and Degree level and for some one decreased the number and for others one increased it: if it were a question of teaching "Why" and "How" the degree people would get a little more "Why" and the Higher National people would get a little more "How"—that was the fundamental difference.

Mr. Wilkins then demonstrated his elastic model which was developed as a visual aid for teaching purposes and details of which, with working drawings, were given on the 5th November, 1954 in "Engineering."

Mr. P. L. Capper (Member of Council) said the paper would be of great value to teachers of structural engineering in the way in which the various methods had been linked together. It was a usual thing for students to learn two or three methods and to look upon them as quite distinct and possibly even as mathematical wangles, but the paper showed how, provided the same assumptions, were made the various methods of approach would give the same answer.

There was one point to which he rather took exception, and that was on page 75, where the author gave the four types of problem and the four requirements for consideration and then stated : " There might also be added the requirement of a method or methods suitable for the class-room, the lecture theatre and the examination hall." Surely the methods which were suitable for practical problems were the ones which should be used in the class-room and the examination hall. It was true that one had to start with problems which were very much simplified, and it was also true that in an examination one had to cram a great deal of work into a limited time. One answer would be to extend the time of the examination, but that, unfortunately, was out of the question on account of student fatigue. The alternative was to simplify the problems, which in fact was done, so that they brought out a knowledge of the principles without involving too much donkey work in the way of computation. It was essential, however, that in their course work students should tackle problems of a more practical nature.

On page 80 of the paper there was something which might or might not be a misprint : the Author referred to the word " Abracadabra " but the spelling in the paper did not agree with the spelling in the dictionary unless, of course, there was some significance in the pronounciation of the word as given in the paper as distinct from the more normal spelling !

Mr. W. MERCHANT (Associate-Member) said he had derived a great deal of pleasure out of reading Dr. Bateman's review of the development of structural theory, and there were many things in it which he had not known before, although he himself had one or two things up his sleeve which possibly Dr. Bateman did not know. For example, he had been taught at M.I.T. before the war to consider whether it was advantageous to use forces or displacements as the unknowns in any particular problem. He had used Dr. Bateman's expression for strain energy some years ago without realising that it had been published and at the moment he had a book in proof containing a statement of the Theorem of Three Moments in terms of fixed end moments which he had fondly imagined to be original.

One facet of analysis to which Dr. Bateman had not drawn sufficient attention in his review was the concept, originally due to Southwell, of group displacements in relaxation procudures. A paper on this subject (51) had already been given to the Institution and some work had also been done in America[85].

It was very difficult to get students to appreciate that one was bound to build in internal stresses, and very often those internal moments with which one started could be of fair magnitude compared with moments due to design loads. That, of course, must be recognised and it did not destroy the necessity for elastic analysis ; in fact, the strange thing was that when one took plasticity into account it was that very fact which caused the need for elastic analysis.

It had been very interesting to see Mr. Wilkins' model which, however, exhibited stability effects and indeed Mr. Wilkins had demonstrated a sway critical load to the audience. The model was not therefore suitable for illustrating Dr. Bateman's paper. In particular the position of the points of contraflexure would not be in agreement with those determined by ordinary elastic analysis. The discrepancy between the model results and calculations might lie here rather than in the use of large deflections.

Dr. E. H. BATEMAN, in reply, thanked the Preside and all those who had contributed to the discussio The President had referred to the difficulty of tracki down early papers, and where, in the present pap reference was made to the work of Otto Mohr developing the slope deflection equations he (t Author) had relied on Johnson, Bryan and Turneau (5) He had tried to verify all the references, but a library where one would naturally expect to fi the Journal of the Institute of Architects of Hanov he had been told that it was one of the journals whi had gone for salvage a year or two before the w He hoped that any librarian who might in the futu have to dispose of old journals of this kind would gi the Institution a chance to look over them.

Professor Baker had referred to the well kno Müller-Breslau equations. He had himself been i troduced to them by Professor Cyril Batho when first went to Birmingham in 1930 and where they h been taught since 1924, but Batho had used thc equations solely for the triangulated structure a had preferred the methods of Otto Mohr for the fram

Dr. Wood had asked what was the advantage strain-energy methods when, if one took the strai energy equation just for one member of a structu and differentiated that one would get slope deflecti equations. In his own experience the advanta was that one added up all the strain-energy componen for the whole structure before one started the d ferentiation, whereas with the slope deflection equ tions applied individually there was the possibili of making mistakes with the plus and minus signs a the labour of the eliminating the unwanted quantiti

Dr. Goldstein had suggested that there was ve little difference in principle between the differe methods, and that, of course, was perfectly true. I had gone on to suggest that as the carpenter h special tools for particular jobs so the structu engineer should have, too. With that he fully agree but the exception he took to the multiplicity of metho was that so many of them purported to be gene methods of attack and he wished that when someb had a bright idea and did something new he wou say that he had obtained an answer to a special pro lem. He commended the example of Sir Richa Southwell who had not claimed the invention of hundred different methods when he had solved hundred different problems by relaxation metho To take a case in point, from the particular field of t present context three years ago Arthur Bolton h devised a very interesting solution to a special proble but it was presented as being a new method of gene application.

With regard to Mr. Capper's remarks he had adm tedly felt a little guilty about suggesting that the was a difference between classroom methods a others : but, from his own experience, he thoug that some university teachers were not interested methods devised for complicated problems if su methods did not also give quick answers to trivi problems.

Mr. Merchant had made an important contributi to the discussion, and he (the Author) was mc interested to hear that Mr. Merchant was likely be the first textbook writer to give a comprehensi statement of the Theorem of Three Moments.

The PRESIDENT thanked Dr. Bateman for replyi to the discussion and the proceedings then terminate

Written Contributions.

Professor J. W. H. KING (Member) wished call attention to the great ease of solution of mu

an continuous beams, and portal frames, both iform and non uniform, which could be achieved by e use of characteristic points. Unfortunately the ferences 43 and 44 did not give the most effective ethod of using them. A brief exposition was given Professor King in the STRUCTURAL ENGINEER (May 50), wherein references for more detailed study uld be found if required[86].

He felt also that the virtual absence of mention of direct analysis methods, both theoretical and ex-rimental, was a great pity, as they presented a very werful method of dealing with problems which ere virtually too difficult or too laborious for practical lution by direct analysis methods. References for ese were also given in the above paper, where again brief outline of the methods appeared.

Professor King was in full agreement with the opinion at it is unwise to adhere slavishly to one method solution—there simply is not *a* best method. A actice he deprecated most strongly was typified by ttempts which have been made to bring Moment istribution and its essentially simple technique back to the fold of mathematics. He could remember at ast two instances where this was tried with continuous eams. In one case the Author went to considerable ouble to show that the moments at supports could e represented by the sums of several series, which e determined and evaluated for various particular ases. This work of art occupied about half a dozen rticles in a well known periodical some years ago. he final tabulated values could have been obtained y Hardy Cross directly, far more quickly and neatly. Another attempt showed that the out of balance t any support had to be wiped out by release there, lus certain fractions of the out of balance at adjacent upports, and thus produced equations for conditions t each support. Needless to say, these were identical ith those resulting from the Theorem of Three Moments—a retrograde step if ever there was one. his work appeared as a paper read to a well known nstitution in the North.

Professor King, personally, had a great reluctance o permit awkward or long winded mathematics to ncroach on structural analysis where it could be avoided. Recognition that integrals are often capable of inter-retation as volumes of solids, or moments of area ould save much confused figuring in strain energy olutions, as instanced for one or two simple cases in is paper mentioned above.[86] He would strongly urge ll mathematically minded engineers to take pity on heir less erudite brethren, and to find tabular or raphical solutions for their more ambitious efforts as n essential part of their work. With the precedents f Hardy Cross, and Southwell, they surely need not e afraid of loss of prestige by so doing, and they vould certainly earn the gratitude of those who have o use their work subsequently, without the use of ny computer more ambitious than a slide rule. The vork of Bolton on the " no shear " treatment of rigid rames,[51] published in the STRUCTURAL ENGINEER, May 1950 and of Rojansksy and Beth on struts[87] show ow anticipation in the first case, and graphics in the econd can vastly simplify work which he knew rom experience to be difficult for many engineers even n graduation. He would venture to suggest that ew become more skilled in mathematical manipulation hereafter.

Mr. J. MORRIS : As this paper is mainly historical, notable omission in the list of references is Grimm's Secondary Stresses in Bridge Trusses'[88]. Had Dr. Bateman noticed this book he would have seen therein mention of Manderla's Munich Polytechnic prize essay ' The Calculation of Secondary Stresses that occur in Simple Trusses as a Consequence of Rigid Joints'[89] published in 1880. Coming to more recent literature it might have been appropriate to refer to my R.38 Memorial prize essay ' The Stressing of Rigid Jointed Frames[92]' published in June, 1936.

When Hardy Cross first produced his moment distribution method nobody, I was informed, could be found to elucidate the rational foundation of the process. The problem came my way in August 1934, the terms of reference being to ascertain the theoretical basis of the process ; to see if it could be extended to braced frames ; and in particular to ascertain if end-load deflection effects could be taken into account. I was at the time an unestablished junior technical officer on two years probation at the Royal Aircraft Establishment, Farnborough, and was given three months in which to solve the Hardy Cross problem. My R.A.E. Report, entitled ' Loads and Bending Moments in Members of a Plane Braced Frame with Rigid Joints,'[91] was dated December 1934 and was published by the A.R.C. as R. and M. No. 1672 (with 23 March, 1935 as the date of receipt of the report by the A.R.C.). In the preamble to this report I said ' By taking account *inter alia* of the deflections of the members of a braced frame, a general successive approximation process of the Hardy Cross type is developed for determining the end loads and bending moments in the members of any plane frame with rigid joints. In addition, a method is given for obtaining the angular deflections of the joints from the results of this process !

In the Chapter " Hardy Cross Moment Distribution Methods and Iteration " in my " Escalator Method in Engineering Vibration Problems "[92]. I say " The Hardy Cross paper marks a definite epoch in the evolution of methods for the calculation of the secondary stresses on structural frames."

" Hardy Cross introduced a new idea which may be said to be a physical form of iteration in which the iteration is carried out, not with the current values of the variables involved, but with the differences of consecutive current values. Thus instead of obtaining the sequence x_1, x_2, x_3,....x_n etc. where x_n tends to the true value of x, one of the quantities to be found, Hardy Cross obtains the series x_1 $(x_2—x_1)$, $(x_3—x_2)$....$(x_n—x_{n—1})$ etc., in which $(x_n—x_{n—1})$ tends to become smaller and smaller ; and the value of x required is found by summing this series."

In my view the most important advance which has accrued from the Hardy Cross concept is the probing of the properties of simultaneous equations and in particular the rational explanation of the ' ill-condition ' of structural equations in certain circumstances and its effect on successive approximation processes.

In this connection if one ' fiddles ' with residuals such as arbitrarily increasing them before ' liquidation ' then although we may obtain apparent balance this will not be with the original loads as Dr. Bateman seems to imply in his observations regarding means for quickening of slow convergence of the Hardy Cross process. It is only in the case of full iteration that mistakes or arbitrary adjustments are ' ironed out.' This fiddling is a common fallacy as I point out in my Escalator book.

I do not agree with Dr. Bateman's assessment of the Hardy Cross and Relaxation methods and feel

that Timoshenko, as quoted by Dr. Bateman, is nearer the mark in the appraisal of these processes.

Dr. H. GOTTFELDT (Member) : Dr. Bateman tried to guard himself against the likelihood that many members of his audience might feel tempted to parade their own particular hobby horses, by emphasising that his selection of methods of elastic design was by no means comprehensive and that, by combining a number of basic approaches, an almost infinite variety of methods can be established.

I was, however, very glad indeed that, in spite of these precautions, Professor Baker made a determined and enthusiastic plea for what he called Müller-Breslau's equations

$$X_a\delta_{ia} + X_b\delta_{ib} + X_c\delta_{ic} + \ldots = \delta_{oi} \ (1 \leqq i \leqq n).$$

and I fully agree with him that the meaning of these equations—a supported point does not move or a fixed one not rotate—is very much easier to grasp than that of a partial derivative or of the mysterious though undeniable tendency of nature—of which man is part—to do the least possible amount of work.

These equations are not derived from a combination of other basic approaches but are fundamental in themselves. The trick lies, of course, in the splitting up of all deformations on the left hand side of the equations into two factors : one the unknown itself, the other the calculable deformation due to a unit value of the unknown.

I would like to confirm Professor Baker's statement that these equations form to this day the basis of the Continental approach to indeterminate structures even though later methods are by no means unknown. I think they will continue to do so for a long time to come, simply for the reason of their universal applicability, and they may well become even more prominent again in the near future because improved methods of solving simultaneous equations have been developed since Müller-Breslau's time.

His own favourite method was to use determinants but he also developed methods of his own for the common case where each equation contains only a limited number of unknowns, usually three.

Since then the quicker and more elegant matrix solution for the determination of the co-factors β_{ik} has become widely known and the mechanical computors mentioned by Dr. Bateman are presumably even better.

Taking it for granted that these machines can solve sets of simultaneous linear equations provided the coefficients of the unknowns are fed into them I would like to ask Dr. Bateman a few questions about their present capacity and availability :

(1) What is the limit to the number of unknowns these computors can deal with ?

(2) Can the machines deal only with fully numerical examples, so that the answers appear as pure numbers, or can they give the numerical values of the co-factors β_{ik} in the solution

$$X_i = \beta_{ia}\delta_{oa} + \beta_{ib}\delta_{ob} + \ldots . ?$$

In the former case each loading condition would have to be fed separately into the machine while a knowledge of the co-factors would enable the engineer to use them for any number of loadings he may have to deal with without further help from the machine.

(3) Are these machines available to the practising engineer and if so, where, and what would be the cost to him of obtaining the solution of, say for example, a set of twenty simultaneous equations ? Would the cost be proportional to the number of equations, or would it rise more steeply ?

I would like to thank Professor Baker for the compliment he paid me when stating that the equations he favours so much are at present only mentioned at the Imperial College and at the L.C.C. Hammersmith School of Building where I am acting as a part-time lecturer, although I sincerely hope that the company is not quite so limited and select as Professor Baker's statement seems to imply.

The reason for this neglect may well be found in the fact that the present-day student has it hammered into him that the solution of a few simultaneous equations is an almost impossible task ; hence the attempts to extend the method of moment distribution to cases for which it is, as Dr. Bateman has pointed out, quite unsuitable.

The real difficulty of the student—and I speak from many years of experience—lies somewhere else : I have yet to meet one who is capable, without doubt and hesitation, of drawing a few simple bending moment diagrams of the primary structure when loaded successively with unit values of the unknowns, this being necessary in order to find the δ_{ik} values in the above equations. There are other ways, but this method of virtual work was invariably and rightly favaured by Müller-Breslau himself of whom I had the great good fortune—and at the time rather terrifying experience—to be a student.

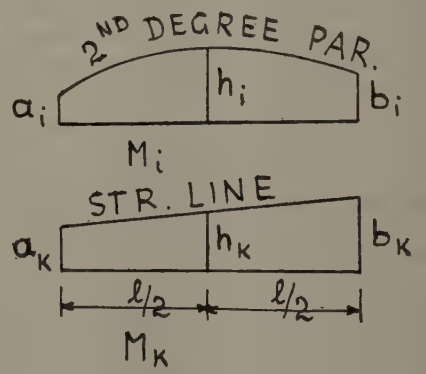


Fig. 4.

The bending moments occuring in $\delta_{ik} = \int M_i M_k d$

are normally of a very simple form and the formu

$$\delta_{ik} = \frac{l}{6}\left(a_i a_k + 4h_i h_k + b_i b_k\right),$$

covering the case in Fig. 4 with all its possible modi cations, will suffice for many practical cases, th eliminating the need for actually performing t integration. Extensive tables of integrals for a wi variety of bending moment diagrams may be four in the chapter on ' Theory of Structures ' in Probst Comrie's Civil Engineering Reference Book[93].

I would like to augment Professor Baker's remarks drawing attention to the value of Müller-Breslau's uations for the determination of influence lines, pecially when coupled with the w—values which introduced for this purpose; tables of these may o be found in the chapter mentioned above. I ink Müller-Breslau was the first to show, and make stematic use of the fact, that the elastic curve of the imary structure when simultaneously loaded with

$= \beta_{1a}$, $X_b = \beta_{1b}$, $X_c = \ldots$. is the influence line for X_1.

Dr. Bateman devotes most of the first page of the inted version of his paper to the confusion that ists at present in our literature about certain tech-cal terms. As to the word 'frame' I would prefer, ntrary to Dr. Bateman, to exclude continuous ams from its range; to my mind the word conjures a picture of something with corners, something two dimensions. The addition of the American rm 'bent,' corresponding more strictly to the German Rahmen,' to our vocabulary may perhaps be useful. On the other hand, this two-dimensional thing may ell be a 'latticed' or 'triangulated' 'frame,' just the closely related 'arch' may be formed of a—ould I say beam?, or joist? or girder?—which has ther a solid web or is formed by triangulation.

An even worse offender is the word 'moment.' am sure that not one student in a hundred when, being confronted with a simply supported beam, first 'takes moments' to find the reactions and ien 'finds moments' at points along the beam is vare of the difference between these moments ne overturning, the other bending), with the result at he is never quite certain whether to consider all ads acting on the beam or only certain of them. Then ere are piers, columns, stanchions, pillars, struts d other terms for a member subject to compression, l with what Sir Ernest Gowers in his 'Plain Words' lls a penumbra of uncertainty.

May I suggest that the Council of our Institution ight consider the desirability of setting up a com-ittee, possibly in collaboration with other interested odies, with a view to issuing a glossary of such ubious terms? In the course of time this might rove just as valuable as lists of standard notations r of standard mathematical symbols. I would be terested in Dr. Bateman's views upon this suggestion.

Finally a few words on Dr. Bateman's very cautious pproach to the plastic theory, an approach with hich I fully concur. To my mind there has always een an element of unreality connected with this heory, simply because I fervently hope that all tructures which I have ever designed will for ever emain in the elastic region.

What the plastic theory has done is to show us that, the hypothetical case of overloading, some types f structure have greater hidden reserves than others, nd of this fact we are, in my view, already making ull use. What other ground is there for allowing a imply supported beam to be calculated for a stress of 0 t/sq. in. while the stress in a tension bar (with no lastic reserve) is limited to 9 t/sq. in.? And for vhat other reason is the stress in a beam on three upports still further increased to 12.5 t/sq. in.?

This latter figure may not look familiar; for some nfathomable reason it hides itself, in our specifications nd by-laws, under the cloak of a fictitious maximum ending moment of $Wl/10$ although we know jolly vell that under the expected working load the bending noment will be $Wl/8$.

It is true that no similar allowances exist at present for frames (or bents?). But is there any reason why frames should be treated more severely than continuous beams? If such allowance were made then the plastic theory will, in my view, have achieved all it can ever hope to achieve. Does Dr. Bateman agree? This is, incidentally, the view widely held on the Continent.

Mr. J. C. DE C. HENDERSON: I would like to add my appreciation concerning the excellent historical research which Dr. Bateman has done. I fully realise that the paper does not pretend to be a comprehensive treatise giving view of all and sundry. I would, however, suggest two very good references to be added to the already long list, namely, "A History of the Theory of Elasticity etc." by Isaac Todhunter [94], and "The History of the Strength of Materials" by S. Timoshenko[95]. The former is out of print, but will be found in most reference libraries.

Regarding the paper itself, I wish to make the following remarks:—

(1) The Theorem of Three Moments. I think this theorem deserves even more emphasis since it was the first type of truly physical canonical form to appear in this subject. Its importance lies in its almost "obviousness" in the mathematical sense and also in its "inevitability." Those who have studied the matrix methods developed by Jenkins, Argyris and others will readily see this. It holds the same position in structural theory as does, for instance, the canonical form of the ellipse in Cartesian coordinates in algebraic geometry. Once this point of view is appreciated, it is not difficult to make up a whole collection of similar theorems relating to other forms of structure. Their detail may be more complex but they all bear the stamp of some invariant form.

(2) I very much agree in the importance to be attached to the classification of structures. I do not think, however, that this paper gives a very clear guide even for the special field considered. Any classification must indicate to the designer the true nature of the structure and give some information as to its particular calculation problems. Bearing this in mind, I suggest that the two sources to be investigated are, (a) Geometry: i.e. the dimensional analysis of the idealised structure to be analysed and the space in which it is embedded. (b) Topological considerations, i.e. those properties which remain invariant under continuous deformation. The classification should be able to be extended to any type of structure whatsoever.

(3) The ill-conditioning of equations is indeed of particular interest. I would suggest that there are two very definite types with which we are concerned in structural theory, namely, (a) apparent ill-conditioning, (b) real ill-conditioning. The literature is well stocked with ideas for tests of conditioning, and no really satisfactory all embracing test appears to have been discovered. However, the normalized determinant of the matrix of the coefficients of the elastic equations gives a reasonably helpful measure. Perfect conditioning is indicated by the value unity and perfect ill-conditioning by the value zero. As an example, consider the beam with constant EI spanning five equal spans. If moments are used as unknowns, the normalized determinant is 0.68, for the support reactions as unknowns this becomes 0.000075. This is only a case of apparent ill-conditioning since both sets of equations are obtainable from each other by non-singular transformations.

If, however, the geometry and releases of a structure turn it into a mechanism, there are no such transformations to be used to enable a solution to be obtained since they will all be singular. This is a case of real ill-conditioning which is itself an invariant.

Professor J. A. L. MATHESON, M.B.E. (Member) : All of us who are interested in the development of structural theory are very much indebted to Dr. Bateman for his scholarly paper ; the list of references alone is invaluable to the serious student. In conjunction with Dr. Hamilton's paper[96] it provides an authoritative account of the growth of the theoretical side of the structural engineer's art.

Although many engineers will regret that their own favourite technique is not singled out for special mention, it is hardly possible to criticise the objectivity of Dr. Bateman's review and many of his comments are very illuminating. The most surprising omission, surely, is any mention of influence lines : perhaps the Author felt these to be outside his terms of reference although the number of papers on influence lines for continuous beams, which Dr. Bateman does deal with, is now very large.

The restriction of the discussion to members of uniform section follows what is unfortunately common practice. Non-uniform members perhaps involve no new principle, but they certainly involve much arithmetic, unless the case is covered by standard tables. The Author's comments on what is a very real problem for the designer would have been of value.

Dr. Bateman is perhaps not as precise as he might have been in his use of the term " strain-energy." The group of theorems and methods in whose title the word " energy " appears have been examined by Williams[97] and, more recently, by Brown[98] : the relationship—and distinction—between these methods is now clear but their nomenclature still leaves much to be desired. This led Niles[99] to discuss ways of improving the descriptions of the various energy theorems, and although he did not by any means dispose of the problem his plea that authors should recognise their responsibility in this matter ought not to pass unheeded.

The genesis of Dr. Livesley's work on digital computation was the thought that the speed of the electronic computer could be put to good effect in an examination of the economy of structural design. The first stage was the development of the programme for the analysis of rigid frames, and this was followed by the attempt to devise methods of automatic design. A procedure for discovering the rigid panel of minimum weight has now been developed and will shortly be described, so that it is now possible to examine the most economical designs for frames of different proportions and under different loading systems : it may then be possible to recognise some general trends which will be of assistance to designers.

Finally the writer would like to sympathise with Dr. Bateman on the precocity of his schoolboy acquaintances : they must be horrid little prigs !

Dr. E. H. BATEMAN, in a written reply, suggested that if he were to interpret his final duty as being to " reply " in detail to all the points raised in the oral and written discussion, his concluding remarks might become longer than the original paper. He would therefore not refer specifically to many matters on which he had been charged with omissions, such as analogue and graphical or semi-graphical methods, because his deliberate intention had been to prese the bare elements of analytical development on t background of an historical perspective, and he want to keep as far as possible within his original terms reference. Moreover to attempt to justify eve expression of opinion would be to abuse the privile of having the last word.

As usual, in such discussions, a point of view e pressed by one contributor was often contraverte by that of another. For this reason he was grate to Professor King for having deprecated " attempts bring Moment Distribution within the fold of math matics," a view which seemed irreconcilable wi that of Mr. J. Morris, the originator of the well kno Escalator Method[92] for the systematic solution large numbers of simultaneous equations. He wou also like to thank Mr. Morris for having added to t record of the paper a reference to that method, whi he had omitted in view of the prospect that the comi availability of E.D.C. facilities would very soon avo the need for formidable manual computations.

He particularly welcomed Professor A. L. L. Baker contribution, since a statement of Müller-Breslau equations, if not easily found in English textbook was now on record in the Journal of the Institutio He would venture however to remark that on pa 52 of Professor Baker's excellent textbook on rei forced concrete[34] these equations appear, witho attribution to Müller-Breslau, as the outcome of application of the principle of virtual work, in t manner of the classic British tradition which ste from James Clerk Maxwell. The same equatio also arise as derivatives of the potential in mc applications of Castigliano's Theorem. Since t points of departure are so close together for each the three systems—the virtual work method used Maxwell, the minimum energy of Castigliano, a the elastic equations of Müller-Breslau—the choi between them would seem to be not so much a matt of principle as of predilection. He could of cour accept unreservedly Professor Baker's assertion th Müller-Breslau's equations " fitted in very well wi the plastic theory."

Nevertheless he felt obliged to challenge the authe ticity of Professor Baker's picture, which had be further illuminated by Dr. Gottfeldt, of continen supremacy in this field, based on faithful adheren to Müller-Breslau. With all respect to the opinio of these two authorities, he could only draw t conclusion that continental engineers had been mu slower than the British to recognise the importan of Hardy Cross. He (Dr. Bateman) had been i troduced to Moment Distribution in 1931 by Profes Cyril Batho, but he had always understood that Cro had been introduced to Batho by Professor J. F. Bak who was first off the mark in this as in other matte Now indeed, after a pretty late start, the continen engineers were following suit. One of the mc competent and far-reaching expositions of Moment D tribution now in print is that of Robert von Halasz[1], the Berlin professor, who had to write the bo three times having twice lost the manuscript throu " the ill winds of war-time fires." This is by no mea the one exception that might prove the postulate Professor A. L. L. Baker and Dr. Gottfeldt. T gospel of Hardy Cross has been spreading rapid on the continent in recent years. For example, Kupferschmid's " Ebene und raumliche Tragwerke[10 we are told by a reviewer[102] that " the ma content of the book is the application of mome

stribution to rigid frames." There are also Rudolf abre's "Das Cross'che Verfahren," now in a third ition revised by Dernedde[103], Barthé's "Grundwerte r das Cross-Verfahren"[104], and from Paris "La éthode de Hardy Cross" by Serge Zaytseff.[105]

The problem to which Mr. J. Morris referred, that secondary stresses in simple trusses with rigid joints, as specially significant as an example of the common round between the designers of airframes on the one and and bridges and civil engineering structures on ie other. This problem had been of great interest nd importance to structural engineers 50 and more ears ago, when the pin joints, such are still to be seen the girders of the Hungerford railway bridge at haring Cross, began to be replaced by riveted con-exions. Bridge designers have long since discounted ie bogey of secondary stresses in trusses of conven-onal form, in the light of calculation, experiment and xperience, although for uncommon designs it might till occasionally be necessary to examine them. He Dr. Bateman) had made such an investigation thirty ears ago in a reassessment of Sir Alexander Rendel's reat cantilever bridge over the Indus at Sukkur, notable structure at the time of its erection in 1886. Ie had estimated the secondary stresses by unsystem-tic relaxation of the incompatibility of deflections nd rotations as first computed by pin-joint theory, aking account of the relative stiffnesses of members neeting at a joint. His predictions of live-load tresses had been closely verified by strain readings nade with the Fereday-Palmer recording extensometer. This was a routine office job which had not been pub-shed in technical proceedings, though there was doubtless still the record of it in the files of the North-Western Railway of India or Pakistan. Incidentally, his early personal experience of agreement between alculation and observation had made it difficult for im to appreciate the view so often expressed to-day hat stresses computed on elastic theories bear no elation to those obtained by experiment—a doctrine vhich seems now to be an essential, if not strictly ecessary, introduction to any paper on design by lastic theory[106]. The simple ad-hoc method of stimating secondary stresses which he had used for Sukkur had of course been codified by Hardy Cross, but this he regarded as an insignificant contribution o structural theory in comparison with the introduction f moment distribution as a method for solving frames vith members transversely loaded between panel oints. Although he did not see eye to eye with Mr. Morris on this particular point, he very much appreciated a contribution from someone who had for so long had a high reputation in aeronautical circles.

He had been asked by Dr. Gottfeldt three specific questions and his answers were

(1) Gabriel Kron[107] had recently described the inversion of a 256 × 256 matrix, which implied the solution of a problem with 256 unknowns, or elimination of 256 redundancies.

(2) A solution could be obtained in the form $X_i = a_{ij} P_i$, where X_i are the required quantities, P_i are applied loads or moments, and a_{ij} is a matrix depending only on the geometry of the structure and the elasticity of the members so that there is a structural solution represented by a_{ij} which is independent of the loading. The matrix coefficient a_{ij} is formed by inversion of a matrix, the order of which is the number of indeterminate quantities. Thus the loading can be varied at the designer's pleasure after the structure has been effectively solved.

(3) One firm of consulting engineers has used "the machine" for an important design, as Livesley and Charlton have recorded,[78] and where one has shown the way others can follow. The cost would rise more steeply than the number of equations, but the ratio would depend on the degree of cross-coupling of the unknowns in the system of equations before inversion.

Dr. Bateman, as chairman of the B.S.I. Sub-Committee on letter symbols and abbreviations for mechanics and structures, warmly welcomed Dr. Gottfeldt's suggestion that the principle of standardisation or codification should be extended to cover the technical terms used in structural engineering, and he hoped that before long this matter would receive attention from the Science Committee of the Institution.

Several points of fundamental interest had been raised by Mr. Henderson, including the difficulty of a satisfactory test for ill-conditioned equations. As Mr. J. Morris had shown,[108] it is sometimes possible to represent a particular structure by either well-conditioned or ill-conditioned equations, according to the choice of coordinates. Thus, in the very simple and elegant illustration designed by Morris, if an ellipse with a large eccentricity is defined by diameters which cut obliquely, much greater accuracy is needed to define the centre than when the orthogonal diameters are used. On the other hand as the eccentricity is diminished and the limiting circle is approached, the degree of possible ill-conditioning must diminish. The advantage of the most favourable choice of coordinates was first recognised by Müller-Breslau, whose great and well-merited reputation rests very largely on the art, ingenuity and skill which he exercised in devising the most economical coordinate systems for particular classes of structures. Müller-Breslau's object was to choose his indeterminate quantities in such a way that as many as possible of his δ_{ik} coefficients had zero values. His contribution in this sense could in his (Dr. Bateman's) opinion could only be fully apprehended by those who had completed a graphical analysis of an actual triangulated structure with at least three statical redundancies.

The omission of reference to members of non-uniform section, to which Professor Matheson had drawn attention, was due not to any lack of appreciation of the importance of this very practical requirement but simply to avoid venturing into deeper waters before the shallows had been thoroughly plumbed. Some years ago he had amused himself by deriving simple expressions for the strain energy in some representative types of variation of cross section, and had obtained some interesting results for structures of low redundancy such as the rigid frame bridge. He felt that he might one day be able to offer these notes for publication, especially now that energy methods had once more been made respectable as a result of the articles by E. H. Brown in Engineering to which Professor Matheson had referred[98] and the rather more ambitious and comprehensive series communicated to the current volume of Aircraft Engineering by J. Argyris[109].

In conclusion, Dr. Bateman wishes to thank all who had contributed to the discussion, and to apologise to members whose questions he had not fully answered

in view of his obligation not only to reply to the discussion but to close it. He particularly appreciated the personal interest shown by the President, Dr. S. B. Hamilton who, from his own unrivalled knowledge of the history of structural engineering had so admirably epitomised the work of Otto Mohr. He (Dr. Bateman) felt very strongly, with the President, the need for an authentic technical biography of Mohr, and not only of Mohr but of other great pioneers of structural engineering theory, in whose work might be seen the origin of all the methods of analysis used in design offices today.

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Book Reviews

(*continued from page 284*)

Heating in Industry and **Ventilating in Industry,** by G. A. Williams. (Richmond, Surrey : Association of Engineering & Shipbuilding Draughtsmen, 1954.) 8¼ in. × 5¼ in. 101 and 36 pp. Price 3s. each.

These are two useful booklets on heating and ventilating industrial buildings. The publication on heating describes and compares the various different heating systems, gravity flow and hot water systems, steam heating, high pressure hot water, heating by gas and electricity. Heat losses from a building are assessed and there are chapters on measurements and thermostatic controls and on radiant heating.

The booklet on ventilation covers natural and mechanical ventilation, types of fans, duct systems, air filters, measurement of air flow, standards of ventilation, air movement and distribution, humidity control and air-conditioning.

Both publications are well illustrated and contain short bibliographies.

The Application of Light Alloys to Structural Engineering, by J. McHardy Young. (Richmond, Surrey : Association of Engineering and Shipbuilding Draughtsmen, 1955.) 8¼ in. × 5¼ in. 43 pp. Price 3s.

This booklet gives first the classification and nomenclature of aluminium alloys and a comparison of their properties, methods of production and design stresses. The principles and practice of design a then dealt with and reference is made to the mo useful applications of light alloys in structural enginee ing. Short notes on corrosion and protection a added and a paragraph on economics. There is a appendix containing useful tables and a bibliograph

Architect's and Builder's Desk Diary and Buildi Legislative Index, 1955. (London : Hadden, Best Co. Ltd., 1955). 8 in. × 5½ in. 103 pp. and de diary.

The object of this Legislative Index is to provide ready means of reference to the principal provisions the statute law and byelaws relating to building building construction, drainage and sanitation, et which are likely to concern architects, building su veyors, civil engineers and builders. The Index cove the relative byelaws and principal statutory provisio in force in London, the building byelaws issued by t Ministry of Housing and Local Government for t Provinces and the main statutory provisions in for affecting buildings and allied matters in the Provinc New sections have been added in this edition coveri provisions relating to Town and Country Planning a Housing, and there is a section on technical inform tion, including tables and a list of technical publicatio and British Standard Codes of Practice for buildin A useful desk diary, with a half-page for each day a a section for addresses and telephone numbers co pletes the volume.

(*continued on page 302*)

Institution Notices and Proceedings

IMPORTANT NOTICE TO SUBSCRIBERS

Subscribers, Newsagents and Subscription Agencies at home and abroad are asked to note that as from the 1st January, 1955, copies of THE STRUCTURAL ENGINEER can only be obtained on application to the Secretary of the Institution of Structural Engineers, 11, Upper Belgrave Street, London, S.W.1.

Subscription Rate

The Annual Subscription, post free, to any part of the world is 44s. 6d.

Single Copies

The price of single copies is 3s. 6d. or 3s. 9d. if sent by post.

In order to fulfil all demands, subscribers and others are requested to state their requirements as soon as possible.

The usual rate of discount will apply.

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1., on Thursday, 23rd June, 1955, at 5.15 p.m., Dr. S. B. Hamilton, O.B.E., M.Sc., B.Sc.(Eng.), A.R.C.S., M.I.C.E., M.I.Struct.E. (President) in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

Students

Bamfield, Eric Michael, of Durban, South Africa.
Bhaskaran, Krishnan Nair, of Johore, Malaya.
Bridge, Richard Howard, of Ashton-under-Lyne, Lancs.
Brown, Robert, of London.
Cannell, Gordon John, of London.
Cooke, Graham George, of Bury, Lancs.
Crossley, Brian, of Irlam, Nr. Manchester.
Dziopinski, Bogdan Stanislaw, of London.
Griffiths, Geoffrey Arthur, of Birmingham.
Hughes, Peter, of Bootle, Liverpool.
Iliasu, Salihi, of London.
Jones, Barry Alfred Champion, of London.
Jones, Michael Sidney, of Uffington, Nr. Shrewsbury.
Jordan, John Cleland, of Wishaw, Lanarkshire.
Lambert, Michael Francis, of Bolton, Lancs.
Lim Lai Keat, of Singapore.
Lubkowski, Janusz, of London.
Mountain, Michael Edward, of Alford, Lincs.
Owen, Charles Vivian, of Newton-le-Willows, Lancs.
Pinkney, Alan, of Hebburn, Co. Durham.
Puddephatt, George Ernest, of London.
Rogers, Joseph Arthur, of Bury, Lancs.
Sito, Jerzy Stanislaw, of London.
Smith, Stanley, of Haydock, Nr. St. Helens, Lancs.
Taylor, Eric Henry Frank, of Urmston, Manchester.
Titchmarsh, Ronald John, of Liverpool.
Venkateswarlu, Varanasi, of Madras, India.
Verwaard, Jan Marinus, of Johannesburg, South Africa.
Worthington, William Napier, of Manchester.
Young, Raymond Brian, of Wellington, New Zealand.

Graduates

Abela, Albert, B.Sc.(Natal), of Bulawayo, Southern Rhodesia.
Apte, Gopal Vishwanath, B.E. Bombay, of Jamshedpur, India.
Bailey, Sidney, of London.
Basson, William Eric Stanley, of Rodley, Nr. Leeds.
Basu, Ramendra Nath, B.Sc.(Calcutta), of Bradford.
Botterill, John Alfred, of Orpington, Kent.
Briggs, Granville Ewart, A.R.I.B.A., of London.
Carling, Peter George, B.Sc.(Eng.) Wales, A.M.I.C.E., of Cardiff.
Chu-Cheong, Clive Kendall, B.Sc.(Eng.) Wales, of London.
Davies, Richard, of Pencoed, Glam.
Foxwell, John, of Leigh, Lancs.
Franks, Arthur, B.Sc.(Eng.) London, of London.
Gala, Shamjee Veljee, B.Sc.(Civil) Bombay, of Gandhidham, Kutch, India.
Graney, Dennis Francis, of Hounslow, Middlesex.
Greaves, Brian Eric, of Sale, Nr. Manchester.
Hague, Douglas James, of Croxley Green, Herts.
Hollingum, Kenneth, of Brighton, Sussex.
Jeffery, Michael Henry, B.Sc.(Eng.) London, D.I.C. of Castle Donington Power Station, Nr. Derby.
Jones, Mervyn, of Llanelly, Carmarthenshire.
Lamb, Peter Robert, B.Sc.(Civil) Birmingham, of Wolverhampton, Staffs.
Larnach, William Job, M.Sc.(Durham), of Durham City.
Leong Yoke Sun, B.Sc.(Civil) Wales, of London.
Mukherjee, Kunjalal, B.Sc.(Civil) Calcutta, of London.
Mukerjee, Sachindra Nath, Hazaribagh, Bihar, India.
Patel, Ramanlal Vallabhbhai, B.Sc. Bombay, B.E. Poona, of Jinja, Uganda, B.E.A.
Purvey, Peter Douglas, of Shifnal, Shropshire.
Rigby, Arthur David, of Cheadle Hulme, Cheshire.
Shephard, Patrick Charles, of Norwich.
Slaney, John Richard, B.Sc.(Eng.) London, of London.
Tantry, Prabhakar Vasudev, B.Sc. Bombay, B.E. (Civil) Poona, of Bombay, India.
Wade, Donald Hugh, M.Sc. Birmingham, of Wath-on-Dearne, Nr. Rotherham, Yorks.
Walker, Donald Ancrum, B.Sc.(Civil) Rand, of Outwood, Nr. Redhill, Surrey.
Wilkins, John Richard, of London.

Associate-Member

Cursiter, William Nisbet, B.Sc.(Civil) Aberdeen, of Singapore.

Associate

Wilmoth, Victor James, B.A.(Cantab.), of London.

Members

Cohen, Sidney Philip, B.Sc. Cape Town, A.M.I.C.E., of Johannesburg, South Africa.
Geleff, Svend, of Nairobi, Kenya.

Transfers

Students to Graduates

Hall, Raymond Sidney, of St. Catharines, Ontario, Canada.
Halliley, Barry, of Batley, Yorks.
Hallows, Peter Marcus, of Wirksworth, Derby.
Lancaster, Robert Mortimer, B.Sc.(Eng.) London, of Leeds.
Staples, Graham George, of Nuneaton, Warwicks.

Graduates to Associate-Members

Canning, Philip John Alexander, B.Sc.(Eng.) London, of Ilford, Essex.
Collinson, Thomas Anthony, of London.
Joshi, Aravinda Vithal, B.Sc.(Civil) Poona, D.I.C., of London.
Kafarowski, Zygmunt, B.Sc.(Eng.) London, A.C.G.I., of London.
Steele, Alan David, of Worcester Park, Surrey.
Topliss, Duncan Geoffrey, of Burton-on-Trent, Staffs.
Wilden, Leonard Kenneth, of Dunedin, New Zealand.
Wilson, William Edward, of London.
Wright, James Stuart, of Eastbourne, Sussex.

Associate-Members to Members

Arbuthnot, Frank Marshall Prescott, of Karachi, Pakistan.
Chou, Gilliam Chinyuan, of Singapore.
Humphreys, James Howard, A.M.I.C.E., of Banstead, Surrey.
Keefe, Harold George, M.I.C.E., of London.
Lee, Tuh-Fuh, of Singapore.
White, Ernest Charles, of Epsom Downs, Surrey.

Members to Retired Members

Giller, Frederick Stanley, of Hextable, Kent.
Waddell, Joseph Finlayson, of London.

Associate-Member to Retired Associate-Member

Hopwood, Albert James, of Ventnor, Isle of Wight.

Re-Admission

Associate-Member

Fernando, Habaraduwa Kudabokolo Melson, of Badulla, Ceylon.

OBITUARY

The Council regret to announce the deaths of Einer Boeck-Hansen, Herbert Hudson Braithwaite, Walter Franchise Clarke, Roland Oliver Counsell, Ethelbert Monk Newell (Members); Norman Cyril James Bennett (Associate); James Gordon, James William Harrold (Associate-Members).

LT.-COL. H. S. ROGERS

It is with deep regret that the Council announce the death of a Founder Member and Past President of the Institution, Lieutenant-Colonel Henry Schofield Rogers, C.M.G., D.S.O., who died at his home at Woking, Surrey, on the 3rd August. He was in his 87th year and until quite recently had taken an active part in the affairs of the Institution, serving on the Council and Committees.

Born in Ontario in 1869, he was the son of Colonel H. C. Rogers of the Canadian Militia. He was educated in Canada at the Royal Military College, Kingston, where he was awarded the Gold Medal for general proficiency and the Sword of Honour for military subject In 1889 he was commissioned in the Royal Engineer and after completing training at the School of Militar Engineering, Chatham, he was posted to India where h served for eight years in the Military Works Service He took part in the Tirah Expedition (1897-98) and th Mahsud Waziri blockade of 1899-1901 and received th thanks of the Punjab Government for his services. In 1903 he was posted to the War Office and served Staff Captain, R.E., in the Directorate of Fortification and Works until 1907, when he was appointed Surveyo of Prisons in the Home Office, a position which he held for 28 years.

On the outbreak of war in 1914, Colonel Rogers wa recalled to the colours and served with the British Expeditionary Force in France. He retired with the rank of Lt.-Colonel in 1919 and returned to the Home Office. On his retirement in 1935, he set up in private practice as a Consulting Engineer.

A Founder Member of the Institution, which came into being as the Concrete Institute in 1908, Colonel Rogers served on the Council from 1913 to 1918 and from 1925 until 1954. During his long association with the Institution he served at some time or other on all its Committees and for some years was Chairman of the Literature Committee. He was elected a Vice President in 1935 and was President for the Session 1938-39.

Gifted with a fine presence and a charming personality, Colonel Rogers inspired affection and respect in all with whom he came into contact and he will be greatly missed by his wide circle of friends.

PRESIDENTIAL ADDRESS

The Presidential Address for the Session 1955-6 will be given by Mr. Stanley Vaughan, B.Sc., M.I.C.E., M.I.Struct.E., on Thursday, 6th October, 1955, at 6 p.m., at 11, Upper Belgrave Street, London, S.W.1.

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, October 13th, 1955

Ordinary Meeting at 6 p.m., when Professor A. J. Ockleston, B.E., Ph.D., D.Sc.(Eng.), M.I.C.E., M.I.Struct.E., will give a paper on "Load Tests on a Three Storey Reinforced Concrete Building in Johannesburg."

Thursday, October 27th, 1955

Ordinary General Meeting for the election of members at 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when a Discussion on Ultrasonic Testing of Concrete will be held. Dr. R. Jones, of the Road Research Laboratory, will open the discussion.

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS—JANUARY, 1956

The Examinations of the Institution will next be held at centres in the United Kingdom and Overseas on January 10th and 11th, 1956 (Graduateship) and January 12th and 13th (Associate-Membership).

REPRESENTATION

The Council have appointed the following Institution representatives:

TY OF BIRMINGHAM EDUCATION DEPARTMENT :
Building Professional Courses Advisory Committee :
Mr. E. R. Deeley (Associate-Member)—re-appointed for a further three years.

PERIAL COLLEGE OF SCIENCE AND TECHNOLOGY :
Concrete Technology Advisory Committee : Mr. Ernest Granter (Past President)—re-appointed for a further two years.

DE OF PRACTICE COMMITTEE FOR CIVIL ENGINEERING :
Drafting Committee—Design and Construction of Reinforced Concrete Structures for Storage of Liquids :
Professor A. L. L. Baker (Hon. Treasurer) ;
Mr. F. R. Bullen (Hon. Editor).

WITHDRAWAL OF PUBLICATION

Stocks of the Institution's Report " Specification for ncrete Pile Driving " are now exhausted, and since uch of the matter contained therein is covered by vil Engineering Code of Practice No. 4—Foundations, e publication is withdrawn.

ENGINEERING MARINE AND WELDING EXHIBITION

The Directors of the Engineering, Marine and Weld- g Exhibition, which will be held at Olympia from the t to 15th September, 1955, have invited all members the Institution to visit the Exhibition on Tuesday, 3th September, 1955. A ticket of admission, avail- ble for two persons, which may be used for the official sit or on any other single day, may be obtained on pplication to the Secretary.

DRURY MEDAL AWARD

The fifth competition for the above award will take lace in 1955. The subject is the design of a mobile rane.

Graduates and Students of the Institution who wish o compete are invited to apply for full details to the ecretary : envelopes to be marked in the top left-hand orner, " Drury Medal Award."

The closing date for the competition is October 1st, 955.

The general conditions of the competition are as llows :—

1. The competition shall be for Graduates and tudents of the Institution of not more than 25 years f age.

2. The subject of the competition shall be a design f a structural character, that is to say, primarily tructural design, not planning.

3. The subject of design and conditions shall be repared and issued biennially by a group of five nembers appointed by the Council.

4. The Literature Committee shall appoint a Jury f not less than five to examine the works submitted nd to interview candidates, if found necessary.

5. In order to show that the work submitted is olely the work of the competitor, the documents sub- nitted shall be countersigned by a corporate member f the Institution, or failing this, shall be accompanied y a declaration on a prescribed form signed by the andidate in the presence of a Justice of the Peace or Commissioner for Oaths.

RESEARCH AWARDS

The Council have instituted a Research Prize Fund, from which awards may be made annually to the author or joint authors of papers describing original research which they have carried out. Research awards may be made for papers read at Headquarters or in the Branches and published in the Journal, or for papers published in the Journal only without being read at an open meeting.

The assessment for such awards will be made annually, but awards will be made only to the contributors of such papers as reach a standard judged by the Literature Committee to be satisfactory.

Work submitted under this scheme must be original and may include any of the following :

(a) investigations of an experimental or analytical character ;
(b) studies of historical or statistical records ;
(c) improvements in principles or methods of construction ;
(d) research into methods of structural engineering and building, the nature and use of plant and the organisation of engineering work ;
(e) any related or combined studies which are deemed by the Literature Committee to be of a research character.

In cases where the research work described in the paper was not the work of one individual, the names of all the collaborators should be given in the paper.

Awards may take any or all of the following forms : A research medal ; a diploma ; a money prize.

Application for consideration for a research award must be made to the Secretary of the Institution, and in preparing papers for reproduction in the Journal, authors must comply with the conditions laid down for all such contributions. Particulars of these conditions may be obtained from the Secretary.

In judging research papers, the following factors will be considered :

(a) the nature of the subject and its conclusions ;
(b) the value of the paper in advancing the science and art of structural engineering ;
(c) the standard of preparation and orderly arrangement of the subject-matter.

Research papers will also be eligible for adjudication for the Institution Medals if they comply with the Regulations governing those awards.

The closing date for the receipt of applications in respect of papers published in the Journal between October, 1954, and September, 1955, is October 3rd, 1955.

LONDON GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : D. E. Capelin, 40, Thornton Crescent, Old Coulsdon, Surrey.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The inaugural meeting of the Session will be held at the College of Technology, Manchester, on Thursday, 20th October, at 7.15 p.m., when the Chairman's Address will be given by Mr. J. H. Morris, M.I.Struct.E.

Joint Hon. Secretaries: J.L. Robinson, A.M.I.Struct.E., 314, Northenden Road, Sale, Manchester ; M. D. Woods, 58, Spring Gardens, Salford, 6.

MIDLAND COUNTIES BRANCH

The Annual Dinner and Ladies' Evening will be held at the Botanical Gardens, Birmingham, on Friday, 8th October.

A meeting of the Branch will be held at the James Watt Memorial Institute, Great Charles Street, Birmingham, on Friday, 28th October, at 6 p.m., when the Chairman's Address will be given by Mr. W. D. Christie, M.I.Struct.E.

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES BRANCH
GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : J. E. Jeffries, 18, Radnor Road, Handsworth, Birmingham, 20.

NORTHERN COUNTIES BRANCH

A meeting of the Branch will be held at the Cleveland Scientific and Technical Institution, Middlesbrough, on Tuesday, 4th October, when the Chairman-Elect, Mr. E. A. Parsons, M.I.Struct.E., will give an address entitled " Relationship between Structural and Civil Engineering."

The installation of the Chairman will take place at the Northern Gas Board Demonstration Theatre, Newcastle upon Tyne, on Wednesday, October 12th, when Mr. E. A. Parsons will give his address, " Relationship between Structural and Civil Engineering."

The meetings will commence at 6.30 p.m., preceded by buffet tea at 6 p.m.

Hon. Secretary : O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following Honorary Officers and Committee members have been elected for the Session 1955-56 :

Chairman : Mr. R. Ferguson.

Vice-Chairman : Dr. N. Agnew.

Immediate Past Chairman : Mr. J. C. Malcomson.

Hon. Auditors : Mr. R. Montgomery, S. O. Morton.

Hon. Secretary : Mr. A. H. K. Roberts, " Barbizon," 26, Dunlambert Park, Belfast.

Hon. Assistant Secretary : Mr. J. McClure.

Committee : Messrs. J. H. Currie, W. A. Plester, T. A. N. Prescott, J. M. C. Tyack.

The Opening Meeting of the Branch, " The Chairman presents . . . " arranged by the Chairman, Mr. Robert Ferguson, B.A., B.E., M.I.C.E., M.I.Struct.E., will take place at the College of Technology, Belfast, at 6.45 p.m. and will be preceded by tea at the Overseas League premises at 6 p.m.

SCOTTISH BRANCH

The Opening Meeting will be held at the Ca'doro Restaurant, Glasgow, on Monday, 10th October, 1955, at 6 p.m., when the Chairman's Address will be given by Professor W. T. Marshall, B.Sc.(Eng.), Ph.D., A.C.G.I., D.I.C., M.I.C.E., M.I.Struct.E.

The Annual Dinner and Dance will be held at Burlington House, Bath Street, Glasgow, on Tuesday, 11th October.

Hon. Secretary : W. Basil Scott, M.I.Struct.E., 19, Waterloo Street, Glasgow, C.2.

SOUTH WESTERN COUNTIES BRANCH

The Opening Meeting will be held at the Duke of Cornwall Hotel, Plymouth, on Friday, 5th November, at 6.30 p.m., when the Chairman's Address will be given by Lt.-Colonel R. Hazzledine, O.B.E., T. M.I.Struct.E. The meeting will be followed by informal dinner.

Joint Hon. Secretaries : E. W. Howells, M.I.Struct. 10/12, Market Street, Torquay, Devon. C. J. Woodr J.P., " Elston," Hartley Park Villas, Tavistock Ro Plymouth.

WALES AND MONMOUTHSHIRE BRANCH

The Opening Meeting will be held at Swansea Tuesday, 25th October, when the Chairman's Addre " Recent Trends in the Design of Industrial Building will be given by Mr. A. V. Hooker, M.I.Struct. A.M.I.C.E.

The Address will be repeated at Cardiff on Tuesd 1st November, and at Colwyn Bay on Saturday, 1 November.

Hon. Secretary : K. J. Stewart, A.M.I.C. A.M.I.Struct.E., 15, Glanmor Road, Swansea, Gla

WESTERN COUNTIES BRANCH

Hon. Secretary : E. Hughes, M.I.Struct.E., 23, Sou down Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The Opening Meeting will be held at the Gr Northern Hotel, Leeds, on Wednesday, 19th Octo at 6.30 p.m., when the Chairman's Address will be gi by Professor R. H. Evans, D.Sc., D. es Sc., Ph. M.I.C.E., M.I.Struct.E., M.I.Mech.E.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C. A.M.I.Struct.E., P.O. Box No. 3306, Johannesbu South Africa.

During weekdays Mr. Tait can be contacted in City Engineer's Department, Town Hall, Johannesbu 'Phone 34-1111, Ext. 257.

Natal Hon. Secretary : E. G. Bennett, A.M.I.Struct c/o The Reinforcing Steel Co. Ltd., P.O. Box 49, M bank, Durban.

Cape Section Hon. Secretary: R. Stubbs, M.I.Struct African Guarantee Building, 8, St. George's Str Cape Town.

INSTITUTION LIBRARY

Conditions of Loan from the Library

1. The Library is available to members of the Inst tion only. In special circumstances, facilities r be granted to non-members to use the Library, they will not be allowed to borrow books.
2. Members of the Institution may borrow any b or pamphlet which is in general circulation, and copies of Transactions and periodicals.
3. Rare books and current works of reference are available on loan, but may be consulted in Library.
4. Members may borrow books either by mal personal application or by post.
5. The period of loan is a fortnight, but this period be extended for a further fortnight provided the book is not required by any other member, provided that application for extension is m before the original period of loan has expired. Institution pays all outgoing postage. The m bers are responsible for the return postage.

Loans are only made on the understanding that the borrower will replace any book or periodical that may be lost or damaged through any circumstances connected with the loan, including the period of transit to and from the Library. All publications should be returned by registered post and adequately protected against damage.

The Library is open from Monday to Friday, from 9.0 a.m. to 5.0 p.m. during the Session, and 9.30 to 5.0 p.m. during the Recess.

A Library Catalogue is not available at present.

PAPERS FOR PUBLICATION

The Literature Committee would be glad to consider ers of papers for presentation at the Institution or publication in the Journal.

The following is a summary of the Committee's quirements relating to articles and papers : a copy of e full conditions may be obtained from the Secretary.

(1) Articles must be of an appropriate character, ving a bearing upon structural engineering or upon me kindred scientific or constructional subject, and ust be approved by the Literature Committee. A ort title is an advantage.

(2) Contributions must be original either in subject-atter or in presentation. Articles which have already en published or have been read to other organised dies, or are carelessly prepared, will not be accepted r publication.

(3) The style of writing will necessarily vary with e individual, but authors are requested to write as ainly and simply as their subject will allow. Papers ould be written in the third person.

(4) Where the subject allows, a brief introduction or nopsis should state clearly the purpose and scope of ne paper or article, and the author's conclusions or commendations should be summarised at the end of ne paper.

In order to facilitate the indexing of articles for ference, the author will be required in addition to repare a short precis not exceeding 25 words for clusion under the title of the paper on the contents age of the Journal.

(5) Illustrations are desirable where they assist in xplaining the context or are fundamental to the ubject. They should not be used if unnecessary for nese purposes. Illustrations may be either line drawings or photographs.

(6) Line drawings must be specially prepared for eproduction on smooth white paper, or clear tracing aper, with heavy main lines and large clear lettering rawn in Indian ink with a mapping pen. Alternatively, the author may submit drawings on one sheet f paper with the relevant lettering on a cover sheet of racing paper.

The printed page of THE STRUCTURAL ENGINEER is in. wide by 10 in. deep. The drawings, where practicable, should be prepared not larger than twice this ize with a view to half-scale reproduction. Unavoidably large drawings which require reduction to one-third ize or less, must be specially heavy and with proportionately large lettering for clear reproduction. Ordinary vorking drawings are not satisfactory.

(7) Where photographs are submitted they should e printed *black on glossy paper.*

(8) MS. typewritten in double spacing should be ubmitted in duplicate.

Brevity is an advantage and papers should not normally exceed 7,500 words in length.

REPRINTS FROM " THE STRUCTURAL ENGINEER "

Authors who wish to obtain reprints of their papers published in " The Structural Engineer " should notify the Secretary of the Institution not later than the 15th of the month in which the paper is published.

EXAMINATIONS

Preparation for the Examinations of the Institution by Attendance at Technical Colleges

A candidate for Graduateship or Associate-Membership may be able to attend a technical college ; these notes are intended to guide him in choosing the most suitable instruction.

Preparation for the Graduateship Examination

Technical Colleges offer :

(a) Full-time courses for degrees or Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At technical colleges courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

Preparation for the Associate-Membership Examination

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering ; these cover only part of the requirements for the Associate-Membership Examination.

Colleges in List "A" provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

List "A"

Bath Technical College.
Belfast College of Technology.
Birmingham College of Technology.
Bolton Municipal Technical College.
Bradford Technical College.
Bridgend Technical College.
Chesterfield College of Technology.
Derby Technical College.
Dudley and Staffordshire Technical College.
Glasgow Royal Technical College.
City of Liverpool College of Building.
L.C.C. Brixton School of Building, S.W.4.
L.C.C. Hammersmith School of Building and Arts and Crafts, W.12.
Manchester College of Technology.

Middlesbrough, Constantine Technical College.
Nottingham and District Technical College.
Salford, Royal Technical College.
South-East London Technical College, Lewisham Way, S.E.4.
South-West Essex Technical College, Walthamstow, E.17.
Stafford, County Technical College.
Stockport College for Further Education.
Twickenham Technical College.
Willesden Technical College, N.W.10.

Colleges in List " B " provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete.

LIST " B "

Brighton Technical College.
Cardiff Technical College.
Edinburgh, Heriot-Watt College.
Huddersfield Technical College.
Leeds College of Technology.
London, Battersea Polytechnic, S.W.11.
London, Northampton Polytechnic, E.C.1.
L.C.C. Westminster Technical College, S.W.1.
Newcastle-upon-Tyne, Rutherford College of Technology.
Plymouth and Devonport Technical College.
Preston, Harris Institute.
Rotherham College of Technology.
Wigan Mining and Technical College.
Woolwich Polytechnic, S.E.18.
West Ham College of Technology.

Students are advised to take the organised courses in Structural Engineering where these are available.

Book Reviews

(*continued from page 296*)

Building. The Evolution of an Industry, by P. Morton Shand. Drawings by Charles Mozley. (London : Token Construction Co., 1954.) 9¾ in., 85 pp. incl. pls. text illus. Price 15s.

Many histories of Architecture have been written, but surprisingly enough, few have been written covering the vast building industry. This book is not a text-book but it is a mine of information, and the author has treated his subject in a delightfully informal manner which will appeal to a wide range of readers both specialist and non-specialist.

The book is of course very short for such a vast subject. Nevertheless it forms a very comprehensive introduction and by skilful sub-division of the subject matter into the three sections—Materials, Building Elements, and Methods of Construction—the evolution of the whole industry is covered, and the reader is left with a thirst for more detailed knowledge.

Throughout the ages one of the major preoccupations of the builder has been the solution of structural problems. The underlying principles of structural engineering have been evolved partly by chance or trial and error, and the most outstanding advances have been ably dealt with by the author, from the very earliest times to the present day or from the pyramids to shell vaulting.

In the section on Cement and Concrete, many of the early pioneers are mentioned. In this connection the article by Professor W. Fisher Cassie in the April issue of THE STRUCTURAL ENGINEER is of interest as it describes an early house constructed entirely of reinforced concrete at Newcastle-on-Tyne.

Built in 1865 by a Newcastle plasterer, Wilkinson, it would appear to be the earliest known example of a small house constructed from reinforced concrete.

The section devoted to building organisation is very informative especially in view of the interest being taken today in the wider aspects of management. The author rightly warns that efficiency and economy are synonymous terms vital to the future of the industry. These are represented by the possession of up-to-date plant, the adoption of every advance in structural technique, coupled with the team spirit of the professional specialists, consultants, technicians and the building labour force.

The author is to be congratulated upon writing such an interesting and delightful book which should inspire renewed pride in all those concerned with building today.

It can be recommended for enjoyable reading and the charming pen and wash drawings by Charles Mozley make a pleasant change from the conventional orthographic projections usually associated with building.

N. K.

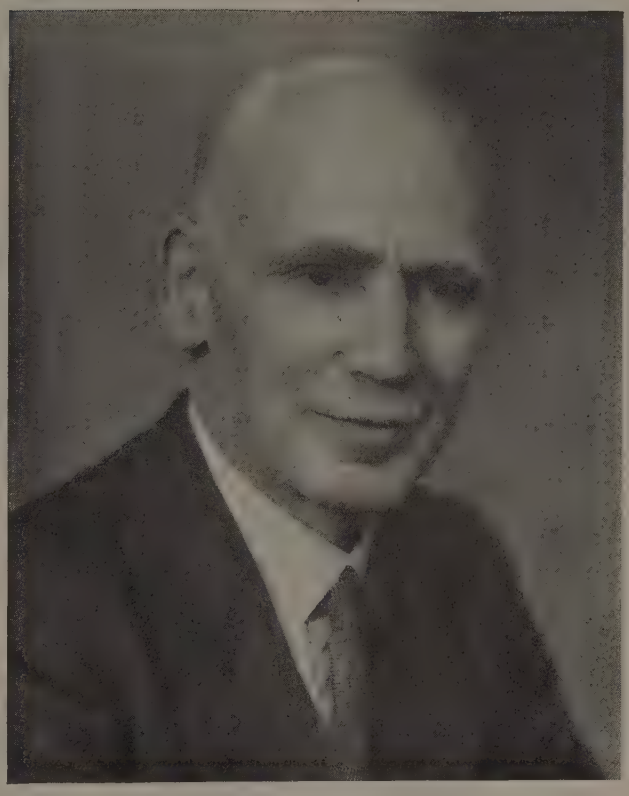
Simplified Design of Structural Steel, 2nd Edition, by Harry Parker. (New York : John Wiley, 195[illegible] London : Chapman and Hall.) 7¾ in. × 5 in. 24 plus viii pp. Price 46s.

This is a practical American book giving data and information for designing many of the structural members occurring in everyday practice. A second edition has been made necessary to bring the book in line with the American Institute of Steel Construction's Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, and to include new formulae, working stresses and design procedures. The tables of properties of structural shapes have been revised and expanded and new safe load tables added. Most of the material in the first edition has been retained, but the discussions and illustrative examples have been brought up-to-date and new items added such as the design of open web steel joists. The methods of computing equivalent distributed loads for use with safe load tables for beams are given and illustrated by examples, and the design of a beam having a long span and relatively light load is discussed. Many new examples and problems for solution have been added to this edition.

The President, 1955-1956

The Institution wel-
mes as its President
r the Session 1955-1956
r. STANLEY VAUGHAN,
no takes office on the
h October.

Mr. Vaughan is a Con-
lting Engineer. Born
London in 1895, he
as educated at Skinners
hool, Tunbridge Wells,
manuel School, London,
d the City and Guilds
Engineering) College,
hich he entered in 1912.
wo years later his
udies were interrupted
the outbreak of war
d the next four years,
14-1918, were spent on
tive service in the
fantry. He was seri-
sly wounded and taken
isoner at the battle of
os in September, 1915.
his release in 1918,
r. Vaughan resumed his
udies at the City and
uilds College where he
aduated B.Sc.(Eng.)
ith second class honours
d obtained the A.C.G.I.
ploma.



Mr. STANLEY VAUGHAN,
B.Sc., M.I.C.E., M.I.Struct.E., A.C.G.I.

In 1919, Mr. Vaughan
ined the Engineering and Public Works Department
Messrs. Trollope and Colls Limited. The following
ear, Dr. Faber started in practice as a Consulting
ngineer and Mr. Vaughan joined him as Chief Assis-
nt. The association has continued ever since, Mr.
aughan later becoming a Partner and the practice
eing styled " Oscar Faber and Partners " (a partner-
ip of six persons).

During these years, Mr. Vaughan has been respon-
ble, either in his capacity as Chief Assistant or as a
artner for the structural (and in some cases also
vil) engineering work on a great number and variety
projects both in this country and abroad, including
hina, Kenya, Tanganyika, Sudan and Egypt. These
orks have included a reinforced concrete jetty, roads
nd bridges, silos, reservoirs, tanks, factories of various
inds (including cement works, steel strip mill, beet
igar factory and artificial silk factory) and also build-
gs of all kinds, such as offices, municipal buildings,
eatre, cinemas, flats, hospitals, schools and churches.
mongst the more notable works have been the House
f Commons rebuilding, the Bank of England, South
frica House, the Royal Horticultural Hall and Lloyds
orporation rebuilding.

In 1939, on behalf of Dr. Faber acting in collaboration
ith Sir Alexander Gibb and Partners, Mr. Vaughan
cted as Executive Engineer and was responsible for
he design and construction of the civil and structural
ngineering work for a large and very urgently required
Ordnance Factory near Liverpool, including over 800 buildings of many kinds, over 20 miles of railways and about 25 miles of roads, two large reservoirs and earthworks. Owing to the size and urgency of the job—the peak labour force exceeded 16,000—the work was dealt with entirely from offices erected on the site. On completion of this project, he acted in a similar capacity on a smaller Ordnance Factory near Nottingham, in this case being appointed Deputy Chief Engineer.

The new President has been associated with the Institution of Structural Engineers since 1930, when he was elected an Associate - Member. He was transferred to full Membership in 1936 and was elected to the Council in 1937. He served on the Council until the outbreak of the second World War, when owing to his work on a succession of " war jobs " in the north of England and other parts of the country, he was unable to give so much time to the activities of the Institution. When he returned to London after the war, Mr. Vaughan resumed active connection with the Institution. He was re-elected to the Council in 1946 and served as an Ordinary Member until 1949, when he was elected Honorary Treasurer, and since 1950 he has been a Vice-President.

In addition to his service as a Member of Council, Mr. Vaughan has given generously of his time and experience in furthering the work of the Institution. He has served on the Committees dealing with Finance, Membership, Science and Research, Bye-Laws and Special Awards, and has been Chairman of the Finance and General Purposes Committee, the Membership Committee and the Science Committee. He has also served on the Civil Engineering Codes of Practice Joint Committee and is a member of the Joint Committee on Vibrated Concrete. He is also a full member of the Institution of Civil Engineers.

Stanley Vaughan is already well known to a large circle of members throughout the country and has made many friends on the occasions when he has visited the Branches of the Institution. During his year of office his kindly and sympathetic personality will win him many more. Members in all grades will wish him well, and those who will have the privilege of serving under him look forward with confidence to a year of continued progress and expansion for the Institution.

Load Tests on a Three Storey Reinforced Concrete Building in Johannesburg*

By A. J. Ockleston, B.E., Ph.D., D.Sc.(Eng.), M.I.C.E., M.I.Struct.E.

Synopsis

Tests made on portions of a ten-year-old reinforced concrete building of normal construction during the course of its demolition are briefly described, and the more important results obtained are discussed.

The deflections and strains for loads within the working range were almost always less than those estimated by normal design methods, and generally agreed more closely with those calculated on the assumption that the concrete was uncracked.

The ultimate loads for failure due to bending or to diagonal tension were greater than design methods would suggest. The plastic theories gave reasonably satisfactory estimates of ultimate loads where failure was due to bending in one direction, but underestimated by a considerable margin the strength of lightly reinforced slabs which failed as a result of bending in two directions.

Introduction

The plans for the remodelling of the Johannesburg Station, which the South African Railways put in hand some years ago, involved the expropriation and demolition of the building housing the Oral and Dental Hospital of the University of the Witwatersrand, a reinforced concrete structure which had been completed in the latter part of 1942, and was scheduled for demolition early in 1952. The Old Dental Hospital (Figs. 1 and 2) was approximately 100 ft. square and was generally three storeys high, with an additional storey over part of the area, but had been planned for subsequent extension to a height of six storeys. The Building By-Laws in force when it was erected were modelled on the 1934 Code of Practice for Reinforced Concrete, and the superimposed loads and permissible stresses for which it was designed corresponded to those in that code.

The demolition of this building provided an unusual opportunity of conducting tests to failure on a modern reinforced concrete structure, and towards the end of 1950, when the co-operation of the Railways Administration had been secured, a committee was formed to organize tests to be carried out concurrently with the demolition. Financial support was received from various interested bodies and most of the equipment required was loaned by the University, the South African Council for Scientific and Industrial Research, and the Railways. The latter organization also agreed to supply the necessary unskilled labour. Further valuable assistance was received from the Building Research Station, Watford; a senior officer visited Johannesburg to advise on the tests and another officer was seconded to assist with the actual testing.

Before the end of 1951 plans for the tests had been drawn up, staff engaged, and the equipment checked and calibrated in the University laboratories. Preparatory work at the Old Dental Hospital began on December 11, and with the co-operation of the Hospital staff much was done during the next month, although parts of the building were still occupied and patients still receiving treatment. The first test commenced on January 15, four days after the Hospital closed down. Thereafter testing proceeded continuously with all available staff until the end of March, by which time the main investigations had been completed. During this period the average number of technical staff on the site was eight, and some sixteen different tests or series of tests were made and more than 20,000 observations of strains and deflections recorded. The few remaining tests were completed in April.

In the tests deflectometers incorporating 0.0001 in. or 0.001 in. dial gauges and simple roller-disc gauges reading to 0.01 in. were used for the measurement of deflections. Strains were generally determined by means of Huggenberger or Galileo gauges of the compound lever type, but in some cases electric resistance gauges were used. Mechanical jacks and proving rings or hydraulic jacks fitted with pressure gauges were used for the application of concentrated loads and, with a system of equalizing beams, for line loads. Floors were uniformly loaded with 45 lb. rail chairs, stacked in a regular pattern to avoid arching. Cribs of railway sleepers were built beneath the floors tested to failure, to prevent complete collapse and to ensure the safety of those working below.

Setbacks due to weather were few, and rain interrupted the work on only two occasions. Dust and rubble from the demolition gave rise to certain difficulties, but throughout the tests ready co-operation was received from the demolition contractor and the Railways staff.

Some of the results of the tests have been briefly described elsewhere,[1] and a rather fuller account is given here.

Properties of Materials from the Old Dental Hospital

When the site tests were completed samples of reinforcement, concrete, and brickwork were taken from undamaged portions of the building in the vicinity of the areas tested. The properties of these materials were subsequently determined in the laboratory, and the values obtained were used in the calculations of deflections, strains, and ultimate loads referred to later in this paper.

* *To be read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1 on Thursday, 13th October, 1955, at 6 p.m.*

Fig. 1.—The Old Dental Hospital, (top) shortly after completion, and (bottom) during the course of the tests

Tests on the reinforcement, which was all round mild steel, gave mean yield stresses varying from 47,800 lb./sq. in. for $\frac{1}{4}$ in. diameter bars to 36,200 lb./sq. in. for bars $1\frac{3}{8}$ in. in diameter. The average ultimate tensile strengths ranged from 62,900 to 68,800 lb./sq. in., and the mean elongations on a gauge length of eight diameters from 30 to 33 per cent. The mean values for the elastic modulus varied from 29.0×10^6 to 31.3×10^6 lb./sq. in.

The properties of the concrete were determined from tests on specimens cut from the samples by means of a diamond saw. The specimens were generally 4 in. square prisms, but specimens of this size could not be obtained from the thinner slabs, and from these prisms of rectangular section were cut, one side being 4 in. and the other somewhat less. It was not always possible to avoid all the reinforcement, and in such cases the cutting was planned so that any steel in the prisms ran in a transverse direction.

The concrete specimens were all tested in an air-dry condition. The densities were determined from the dimensions and weights of the prisms, allowance being made for any steel present. In the tests for modulus of rupture the specimens were simply supported on a span of 12 in. and equal loads applied at the third-span points. Specimens of rectangular section were tested with the 4 in. dimension vertical, to maintain a constant span-to-depth ratio, and those containing reinforcement with the steel in the compression zone and, wherever possible, outside the middle third of the span. The compressive strengths were determined by equivalent cube tests on the broken portions of modulus of

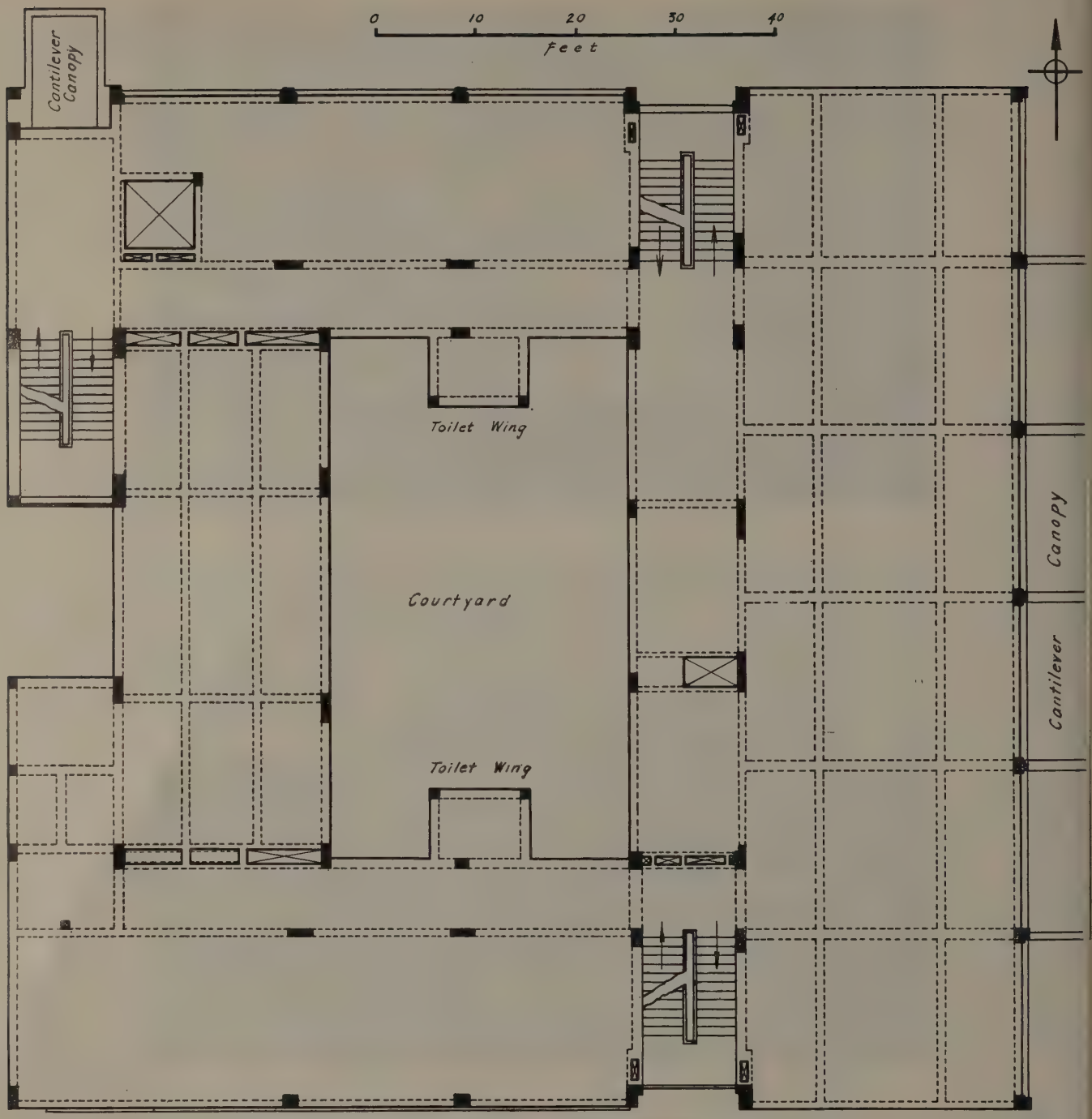


Fig. 2.—Plan of a Typical Floor, the First Floor, of the Old Dental Hospital

rupture specimens or other suitable lengths of prism, specimens of rectangular section being tested with the short dimension normal to the direction of loading.

The elastic modulus of the concrete for stresses up to about a third of the equivalent cube strength was found from tests on 4 in. × 4 in. × 8 in. prisms, the strains over a central 4 in. gauge length being determined by means of a pair of modified Huggenberger Type A Tensometers. When these tests were completed a simpler and less sensitive averaging extensometer of 4 in. gauge length was fitted to each prism and the strains recorded throughout a test to failure. Longitudinal cracks were observed in some of the prisms before the maximum load was reached, and as a rule failure did not occur until extensive cracking had developed and the strains were several times those corresponding to the maximum load.

The results of the tests on the concrete are summarized in Table I. The values given by individual specimens from the same sample did not vary greatly, seldom differing by more than 20 per cent. from the mean, but the mean values for different samples varied considerably although all the concrete tested was to the same nominal mix, 1 : 2 : 4 by volume. No clerk of works was employed during the construction of the Old Dental Hospital and the supervision was intermittent.

The equivalent cube strengths were compared with the strengths estimated from the hardness of the concrete as determined by two different patterns of

TABLE I.—PROPERTIES OF CONCRETE FROM THE OLD DENTAL HOSPITAL

Location	Sample No.	Density (lb./cu. ft.)	Modulus of Rupture (lb./sq. in.)	Equivalent Cube Strength (lb./sq. in.)	Prism Strength (lb./sq. in.)	Elastic Modulus (lb./sq. in.)
Toilet Wings	1	145	825 (1)	6150 (4)	3750 (1)	3.50×10^6 (1)
	2	141	630 (2)	4600 (3)	2200 (1)	2.35×10^6 (1)
	3	146	560 (2)	3800 (5)	2900 (2)	3.25×10^6 (2)
	4	143	605 (4)	3650 (5)	2750 (2)	3.15×10^6 (2)
One-waySlab 2nd Flr. S.	5	147	670 (1)	3300 (2)	2200 (1)	2.75×10^6 (1)
	6	150	895 (2)	6550 (4)	4900 (1)	5.15×10^6 (1)
	7	147	830 (1)	5300 (3)	2800 (1)	3.60×10^6 (1)
	8	148	705 (3)	4900 (4)	4250 (1)	4.00×10^6 (1)
Floor System 2nd Floor, East	9	148	620 (1)	4700 (2)	3750 (1)	3.70×10^6 (1)
	10	149	665 (1)	3500 (3)	2450 (1)	3.55×10^6 (1)
	11	144	540 (2)	4300 (3)	1950 (1)	2.35×10^6 (2)
	12	146	600 (6)	4100 (7)	2650 (1)	3.00×10^6 (2)
	13	148	705 (2)	4800 (3)	3450 (1)	4.25×10^6 (1)
	14	149	580 (2)	4200 (3)	3750 (1)	3.85×10^6 (1)
	15	148	625 (3)	3650 (3)	3600 (1)	3.50×10^6 (1)
Floor System 1st Floor, East	16	146	430 (1)	4600 (2)	3150 (1)	3.00×10^6 (1)
	17	154	765 (2)	7850 (3)	5650 (1)	4.95×10^6 (1)
	18	147	480 (2)	3600 (3)	2450 (1)	2.90×10^6 (1)
	19	148	635 (1)	5700 (2)	3550 (1)	3.20×10^6 (1)
	20	149	390 (1)	6550 (1)	3000 (1)	2.55×10^6 (1)
Various	21	146	660 (1)	4750 (2)		
	22	147	520 (3)	3800 (5)		
	23	147	470 (3)	4350 (5)		
	24	146	640 (2)	5600 (3)		
	25	156	600 (2)	5250 (3)		
	26	152	680 (2)	3850 (3)		
	27	154	765 (2)	5600 (3)		
	28	159	740 (2)	4650 (3)		
	29	152	680 (1)	4900 (2)		

The figures in brackets indicate the number of specimens tested

Schmidt Concrete Sclerometer, using the calibration curves supplied by the makers of these instruments. This procedure had been found to give a reasonably satisfactory prediction of the strength of concrete up to about two months old, but seriously overestimated the strength of the ten-year-old air-dry concrete from the Old Dental Hospital. The results showed a wide scatter, and the strengths predicted by the old and new pattern instruments were respectively about 85 per cent. and 40 per cent. greater than the equivalent cube strengths.

In the site tests the mortar screed with which most of the floors of the Old Dental Hospital were finished generally appeared to act as an integral part of the slab, and tests were therefore made on a few specimens cut from samples of mortar screeding. The specimens, which were necessarily small, gave values of 350 to 620 lb./sq. in. for modulus of rupture, and 2,300 to 4,600 lb./sq. in. for compressive strength.

Samples of the stock bricks used for the wall panels of the building were found to have a mean crushing strength of 930 lb./sq. in. when tested in a saturated condition, and of 1,150 lb./sq. in. when tested air-dry. Compression tests were also made on samples of 4½ in. brickwork from the building. The four specimens tested were each 27 in. long and from four to eight courses high. Two were plastered on one face and the other two on both faces. The values of the elastic modulus of the brickwork ranged from 0.24×10^6 to 0.40×10^6 lb./sq. in., and the compressive strengths from 320 to 460 lb./sq. in. ; these values are based on the overall thickness of brickwork and plaster. The specimens plastered on both faces gave the higher values for elastic modulus, but the lower values for strength.

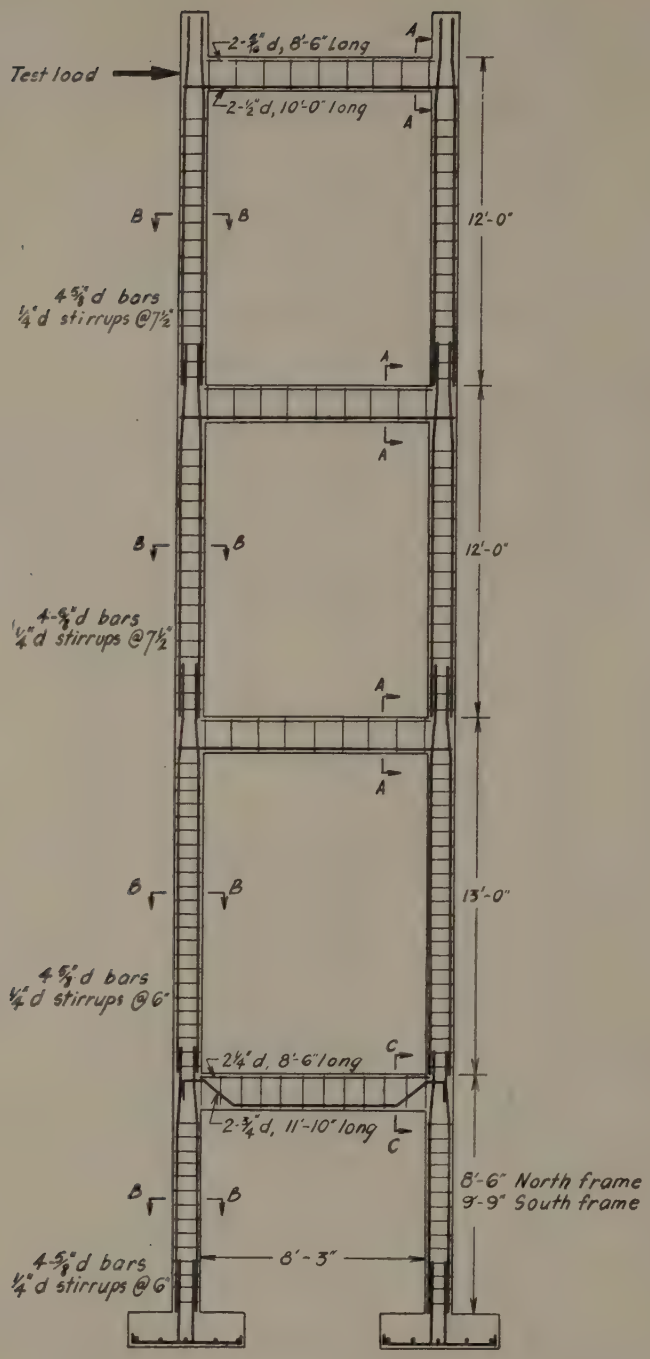
Horizontal Loading Tests on Three-Storey Frameworks

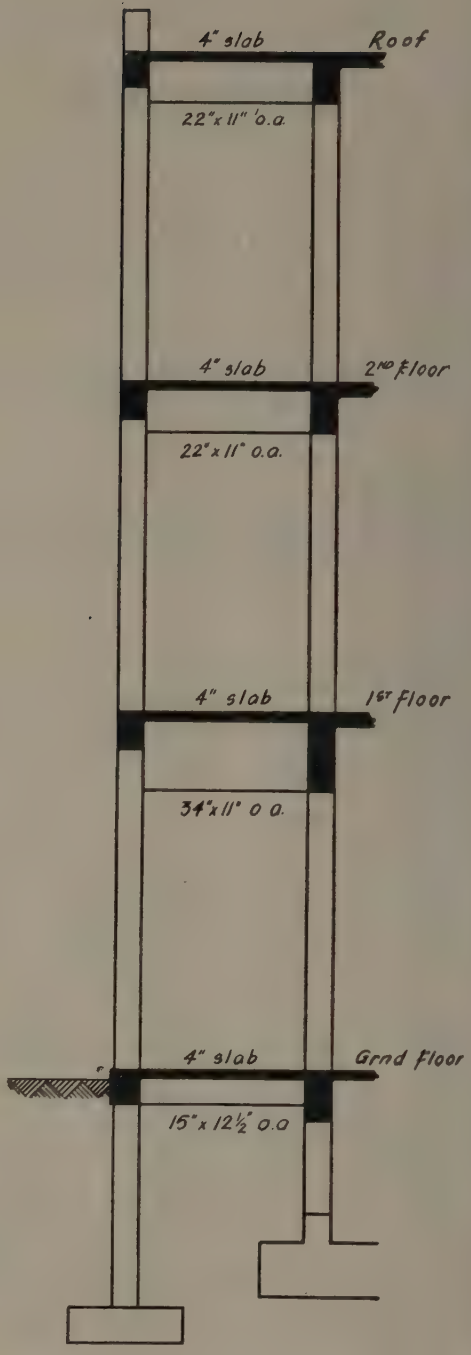
In order to investigate the effect of floors and walls on the strength and stiffness of a reinforced concrete framework tests were made on the two small toilet wings which projected into the central courtyard of the Old Dental Hospital. The wings (Figs. 3 and 4) each consisted of a three-storey single-bay end-frame connected to the main body of the building by reinforced concrete beams and floor slabs ; the walls were all of 11 in. cavity brickwork. The loading adopted for all tests was a single horizontal force applied in the plane of the end-frame at roof level by means of a jack reacting against the main body of the building.

In the first series of tests each wing, with all beams, slabs, and wall panels intact, was subjected to four cycles of loading and unloading, the loads being increased in successive cycles to a maximum of about 38,000 lb. for the north wing, and 57,000 lb. for the south wing. Both wings behaved in the same manner. The beams and slabs connecting the end-frames to the main body of the building acted as horizontal cantilevers, and proved much stiffer than the end-frames themselves. The deflections at roof level where the load was applied were very small, the maximum deflection observed being little more than 0.01 in. for a load of 56,600 lb., and no movement could be detected at the floors below. The load/deflection curves were all nearly linear, and all the tests on the north wing and the first two tests on the south wing gave approximately the same load/deflection relationship. The deflections for the last two tests on the south wing were about 25 per cent. greater than those for the previous tests, but no cracking was observed in this wing. In the north wing a few hair cracks had formed between the members of the end frame and the brick panel in the top storey.

The beams, slabs, and walls connecting the end-frames to the main body of the building above ground level were then cut through and further tests made on the end-frames, the brick infilling of which had been left intact. For the south frame the loads were limited to about 5,000 lb., but the north frame was tested to failure.

The tests on the south frame and the initial portion of the first test on the north frame gave similar results. The load/deflection curves for both frames showed discontinuities at a load of 2,000 to 3,000 lb., roughly the load at which the maximum tensile stresses due to the test load equalled the compressive stresses due to self-weight. Thereafter the deflections again increased linearly, but at a more rapid rate. The deflections were approximately proportional to their height above the ground floor, and the deflections at the roof for loads of less than 2,000 lb. were about twice those





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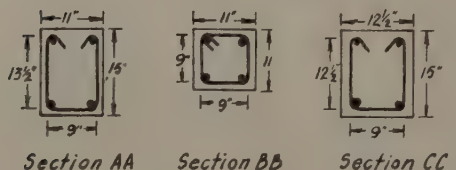

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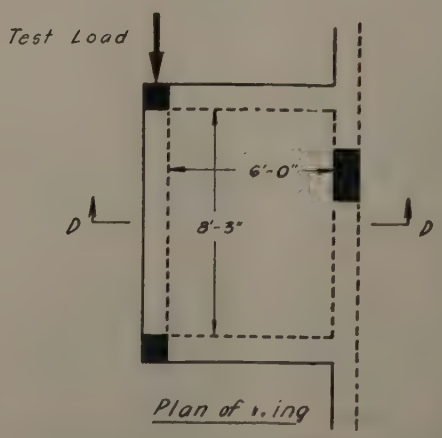
Details of End-Frame



Plan of wing

Fig. 3.—General Arrangement of Toilet Wings and Details of End-Frames

Fig. 4.—Horizontal Loading Test on North Wing in Progress

estimated by treating the frame as a vertical cantilever with a web of brickwork.

In the first test on the north frame, in which a maximum load of about 11,000 lb. was applied, a change in the load/deflection relationship was observed when the load was a little less than 9,000 lb. (Fig. 5). The deflections then increased more rapidly, and shortly afterwards cracks were detected near the bottom of the column in which tension was induced by the horizontal load, and in the adjacent brickwork. In the next test the deflections increased uniformly until, at about 10,000 lb., further cracking developed. At about 19,000 lb. a crack at the construction joint in the tension column at ground floor level began to open rapidly as a result of bond failure at the splice in the column reinforcement just above the construction joint, and the lower panel of brickwork began to lift away from the ground floor beam. When the maximum load of 20,400 lb. was reached the crack extended right along the ground floor beam and into the second column (Fig. 6). With further deflection the load decreased slightly. In a final test the deflection at roof level was increased to about 6 in., but the maximum load attained was rather less than in the previous test. When the concrete was subsequently chipped away it was found that the $\frac{5}{8}$ in. diameter column bars had been lapped about $10\frac{1}{2}$ in. at the splice. Calculations indicated that the maximum bond stress developed during the test had been nearly 700 lb./sq. in.

A final series of tests was made on the end-frame of the south wing after the brick panels had been removed. The load/deflection curves obtained in the first two tests, in which the maximum loads were less than 1,400 lb., were all nearly linear, but in the third test

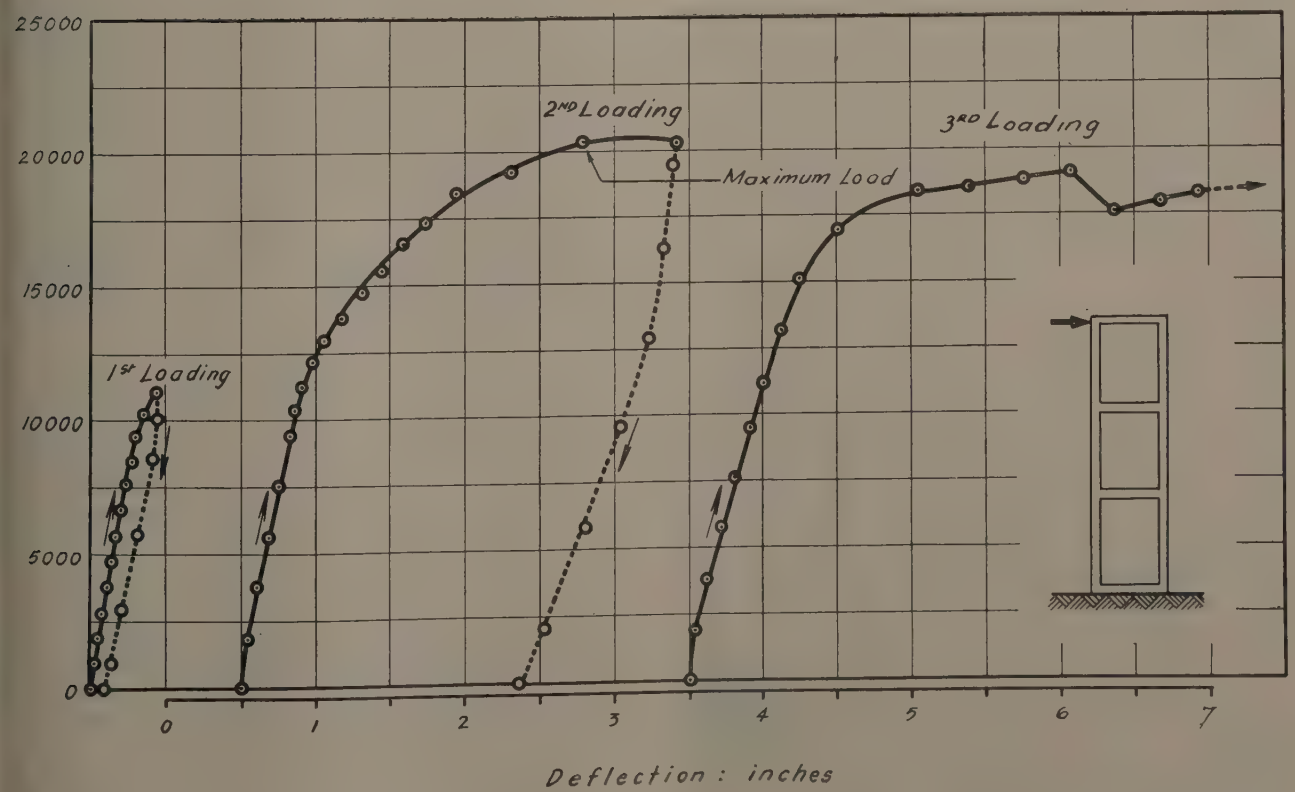


Fig. 5.—End-Frame with Brick Panels : Deflection at Load for Successive Loadings

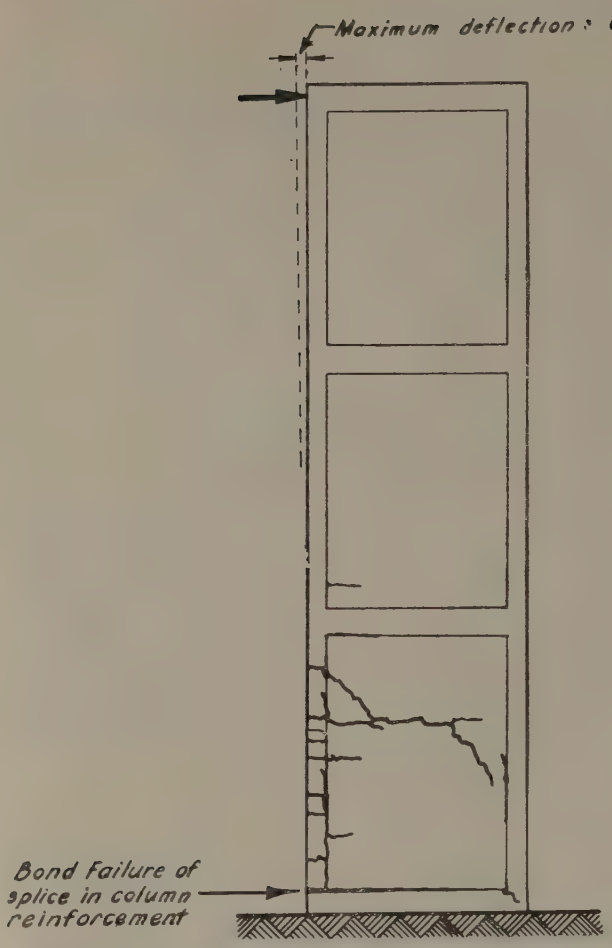


Fig. 6.—End-Frame with Brick Panels : Cracking of Frame and Brick Panels at Failure

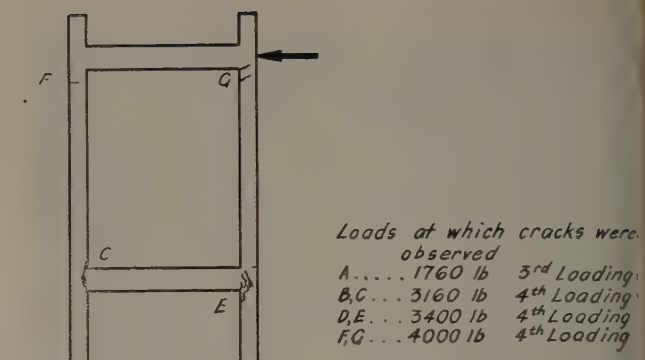


Fig. 8.—Bare End-Frame: Cracking of Frame at Failure

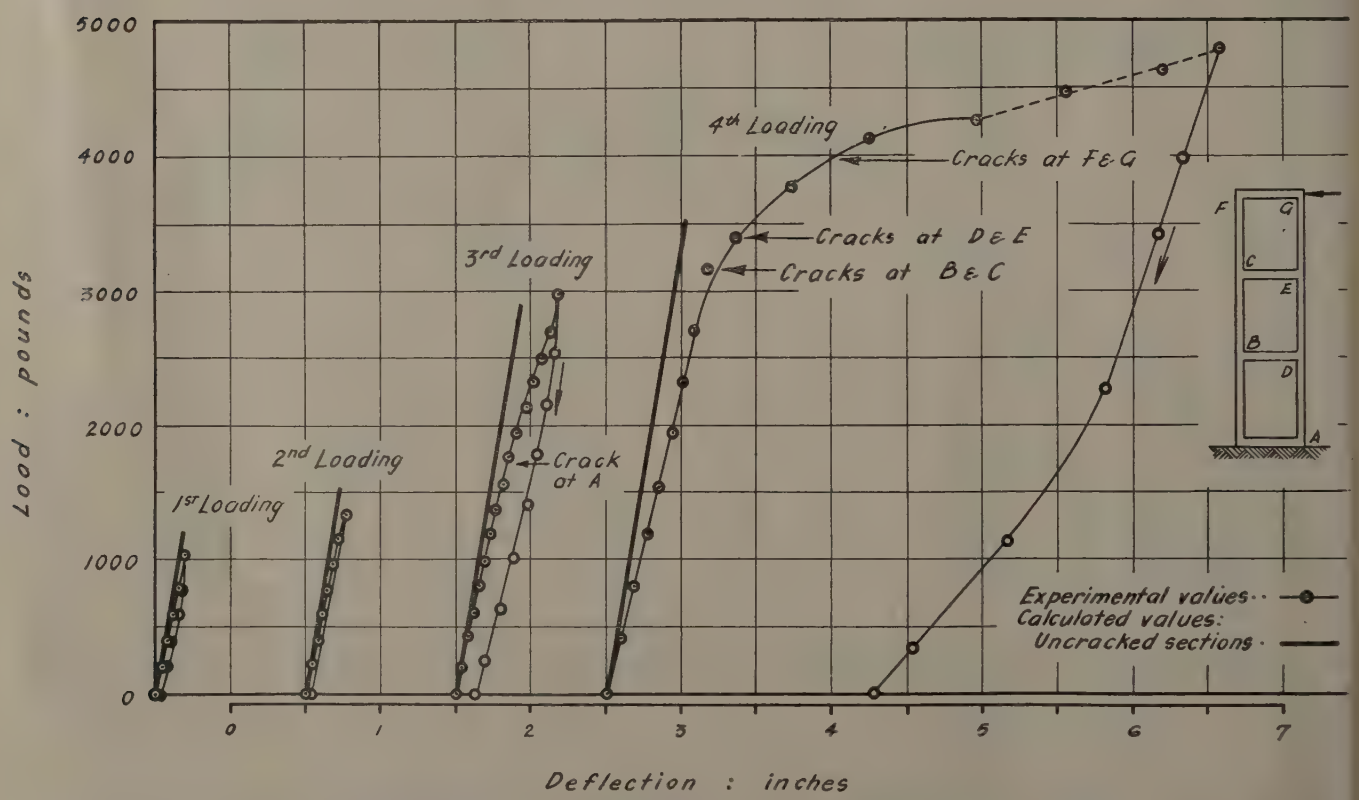


Fig. 7.—Bare End-Frame : Deflection at Load for Successive Loadings

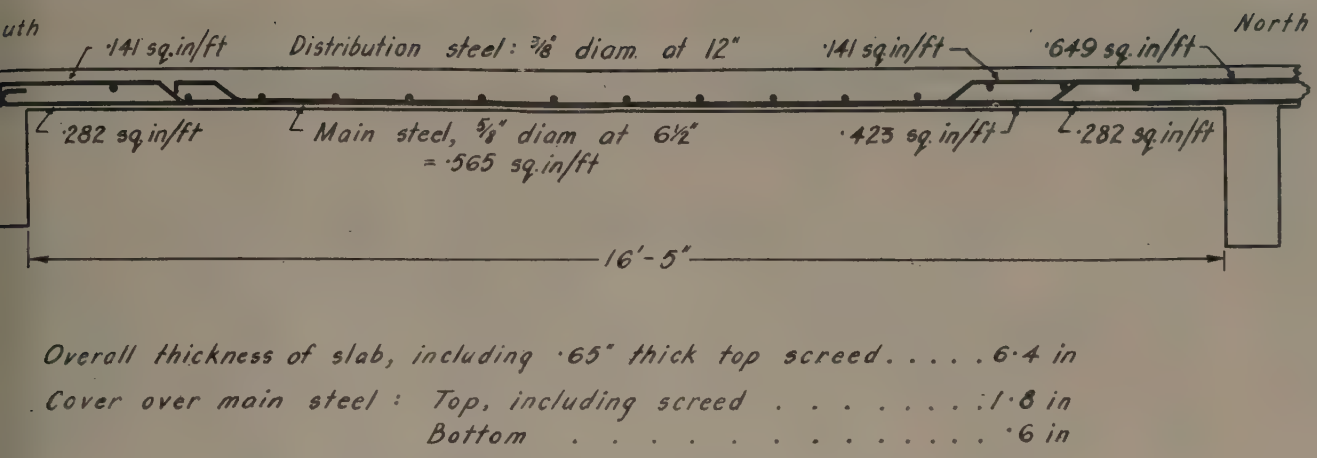

uth
North
·141 sq.in/ft
Distribution steel: ⅜" diam. at 12"
·141 sq.in/ft
·649 sq.in/ft
·282 sq.in/ft
Main steel, ⅝" diam at 6½"
= ·565 sq.in/ft
·423 sq.in/ft
·282 sq.in/ft
16'-5"
Overall thickness of slab, including ·65" thick top screed . . . . 6·4 in
Cover over main steel: Top, including screed . . . . . 1·8 in
Bottom . . . . . ·6 in

Fig. 9.—Dimensions and Details of Reinforcement of One-Way Slab

which loads of up to 3,000 lb. were applied, a change the load/deflection relationship occurred at about ,400 lb., and at 1,760 lb. cracking was observed at the onstruction joint at the foot of the tension column oint A, Figs. 7 and 8). In the last test the deflections ncreased uniformly for loads up to nearly 3,000 lb., nd then began to increase more rapidly. Cracks eveloped between the upper portions of the first-floor nd second-floor beams and the columns (points B and) at 3,160 lb., between the lower portions of these eams and the columns (points D and E) at 3,400 lb., nd in both columns at their junctions with the top eam (points F and G) at 4,000 lb. When a load of bout 4,300 lb. was reached the frame had deflected pproximately 2½ in. and the load/deflection curves ere nearly horizontal.

As a safety measure several reinforcing bars between he frame and the remainder of the building had been eft intact. Initially these had little effect on the rame but at this stage they began to restrict its move- ent, as indicated by the rising portion of the load/ eflection curve shown in broken lines in Fig. 7. It is onsidered that but for these bars the deflections would ave increased with little change in load until failure ccurred in the compression column at ground floor evel, and that the collapse load for the frame may with ittle error be taken as 4,300 lb.

The deflections calculated on the assumption that he frame was uncracked were about three-quarters of he observed values. The calculations indicated that racking would first develop, at point A, at about ,000 lb., and that the collapse load corresponding to he formation of hinges at the top and bottom of each olumn and at both ends of the beams at the first and econd floors would, owing to the reduction in the oments in the beams which would occur when the oncrete cracked, be rather less—about 3,500 lb. robably owing to poor adhesion at the construction oint at point A, the first crack formed at less than half he predicted load, but the collapse load was about 5 per cent. greater than the estimated value.

The results of these tests indicate the extent to which floors and walls can affect the stiffness and strength of a framework. At roof level the deflections of the complete wing, the frame with brick infilling, and the bare frame were in the ratio of approximately 1 : 130 : 1,200. The collapse load for the bare frame was 4,300 lb., the frame with brick infilling failed at 20,400 lb. as a result of local failure at a splice, while the complete wing withstood nearly 57,000 lb. without any sign of failure.

Tests on a Slab Spanning in One Direction

Several series of tests were made on a large panel of slab on the south-west of the second floor, which had a clear span of 16 ft. 5 in., a length of 62 ft., and a nominal thickness of 6 in. The slab was continuous over the beam along its north edge but did not extend beyond those at the other three edges, and had been designed as a slab spanning in one direction for a superimposed load of 93 lb./sq. ft. The dimensions and reinforcement details are shown in Fig. 9; the values given for the thicknesses of the slab and screed, and for the covers over the reinforcement are the means of values measured at a number of points in the slab.

In the first tests a single concentrated load having a maximum value of from 4,500 to 6,000 lb. was applied to the slab through a 3 in. diameter loading disc at various points along the centre-line and each quarter-span line. In all cases the load/deflection relationship was approximately linear. The deflections were small and were generally intermediate between those calculated by the elastic plate theory[2] for the slab with fixed and with simply supported edges; in the calculations the concrete was assumed to be uncracked and the mortar screed treated as an additional thickness of concrete. The strains in the main steel at midspan usually agreed reasonably well with the calculated values except in the vicinity of the load where, probably owing to local cracking, the experimental values were greater than those calculated for the uncracked slab.

Some of the tests were repeated after the slab had been cut away from the beam at the west end. For loads close to the free edge the deflections and strains were much greater than in the previous tests, but there was little difference if the distance of the load from the free end was more than about two-thirds of the span. This is shown in Fig. 10, which gives the widths of slab over which the concentrated loads could be considered to be uniformly distributed in the direction parallel to the supports. These effective widths were determined by comparing the deflections for the concentrated loads with those obtained when a 10 ft. 6 in. length of slab, separated from the remainder of the panel by transverse cuts, was subjected to line loads parallel to the supports.

This 10 ft. 6 in. length of slab was also tested under uniform loading. Within the working range no cracking was detected, and the bending moments derived from the observed strains, on the assumption that the slab was uncracked, corresponded to restraining

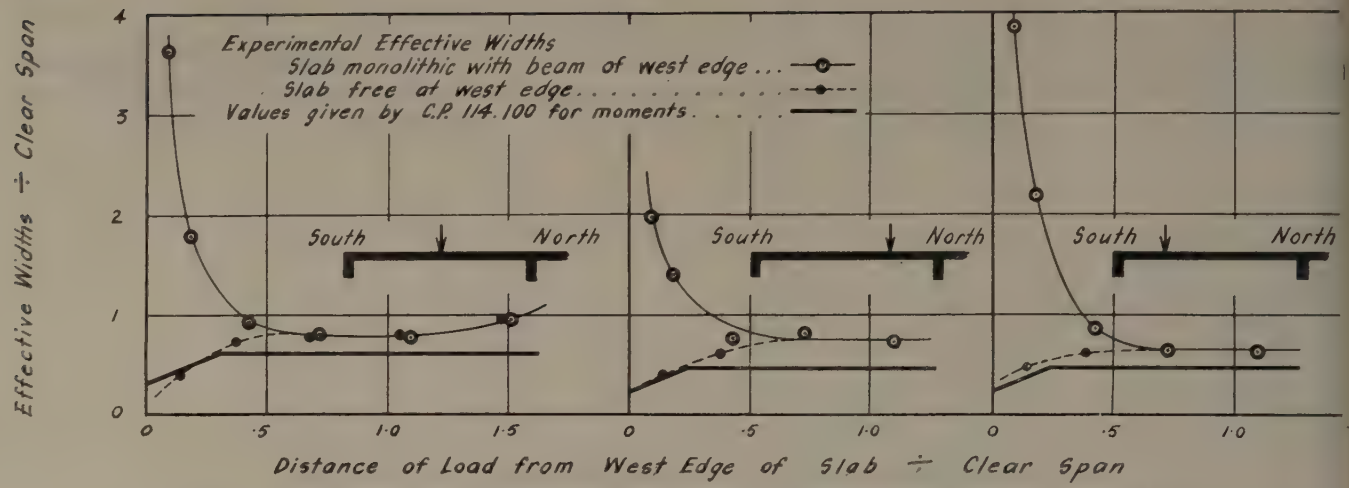

Effective Widths ÷ Clear Span
Experimental Effective Widths
Slab monolithic with beam of west edge
Slab free at west edge
Values given by C.P. 114.100 for moments
South
North
Distance of Load from West Edge of Slab ÷ Clear Span

Fig. 10.—One-Way Slab : Effective Widths for a Concentrated Load on Centre-Line, and on each Quarter-Span Line

moments of 69 per cent. of the fixed-end moment at the north support, over which the slab was continuous, and 54 per cent. of this value at the south support. The calculated deflections for the uncracked slab corresponding to this degree of fixity agreed satisfactorily with the experimental deflections for both the uniform and the line loads.

When this length of slab was tested to failure under uniform loading hair cracking was first detected over the south support at a load rather greater than the design load. Similar cracks later formed over the north support and on the underside of the slab at midspan. With increased loading the cracks at the supports extended, and further cracking developed on the underside of the slab. The reinforcement yielded first at the supports and later at midspan, and the slab failed when the concrete crushed along these sections. The ultimate load of 472 lb./sq. ft. (555 lb./sq. ft. including self weight) was nearly three and a half times the total design load. The calculated collapse load corresponding to the formation of plastic hinges along the supports and across the slab near midspan was 95 per cent. of the experimental value, and the calculated position of the central hinge was close to its actual position. Some of the load/deflection and load/strain curves obtained in this test are shown in Fig. 11. T

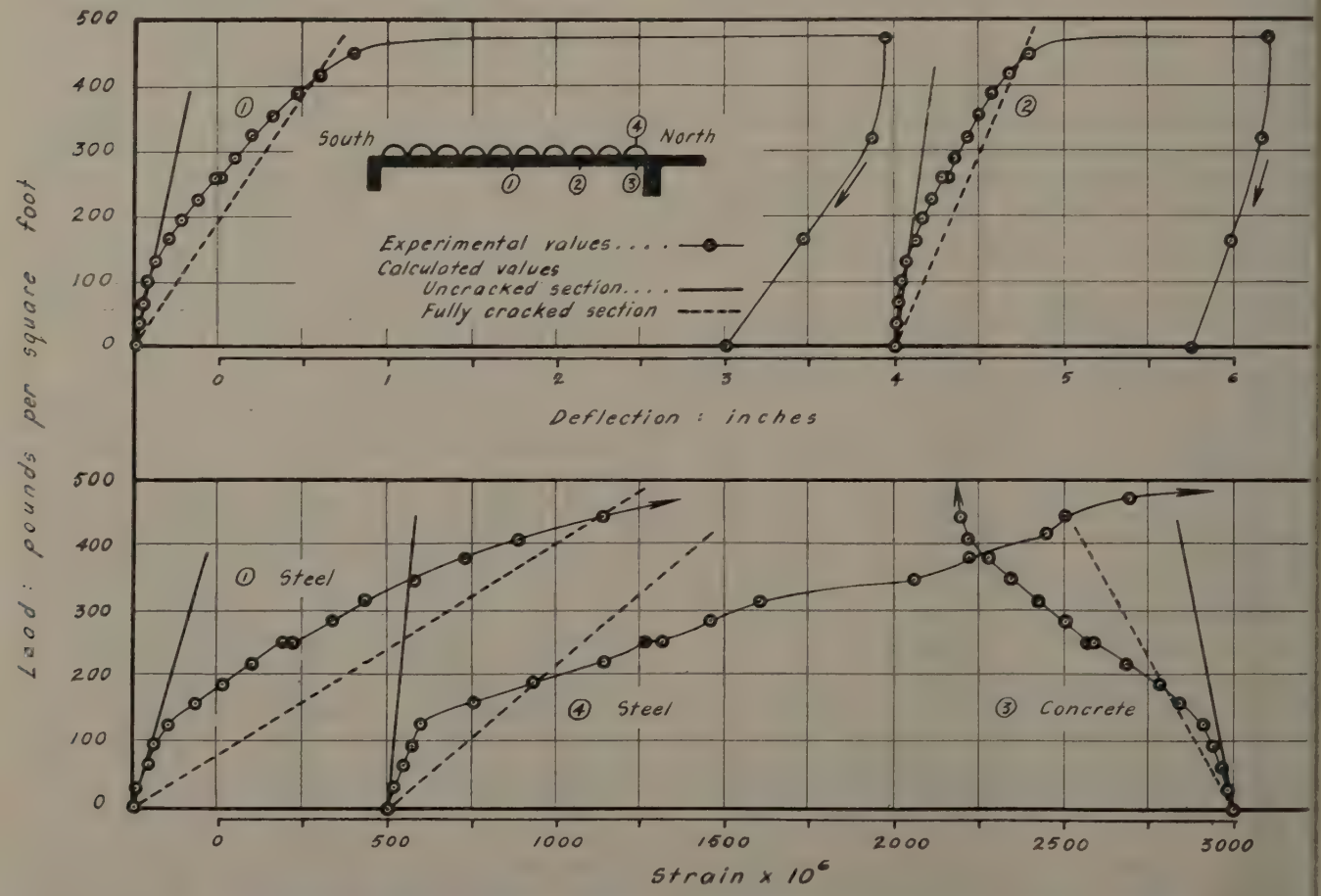


Fig. 11.—One-Way Slab, Uniform Loading : Deflections and Strains for Test to Failure

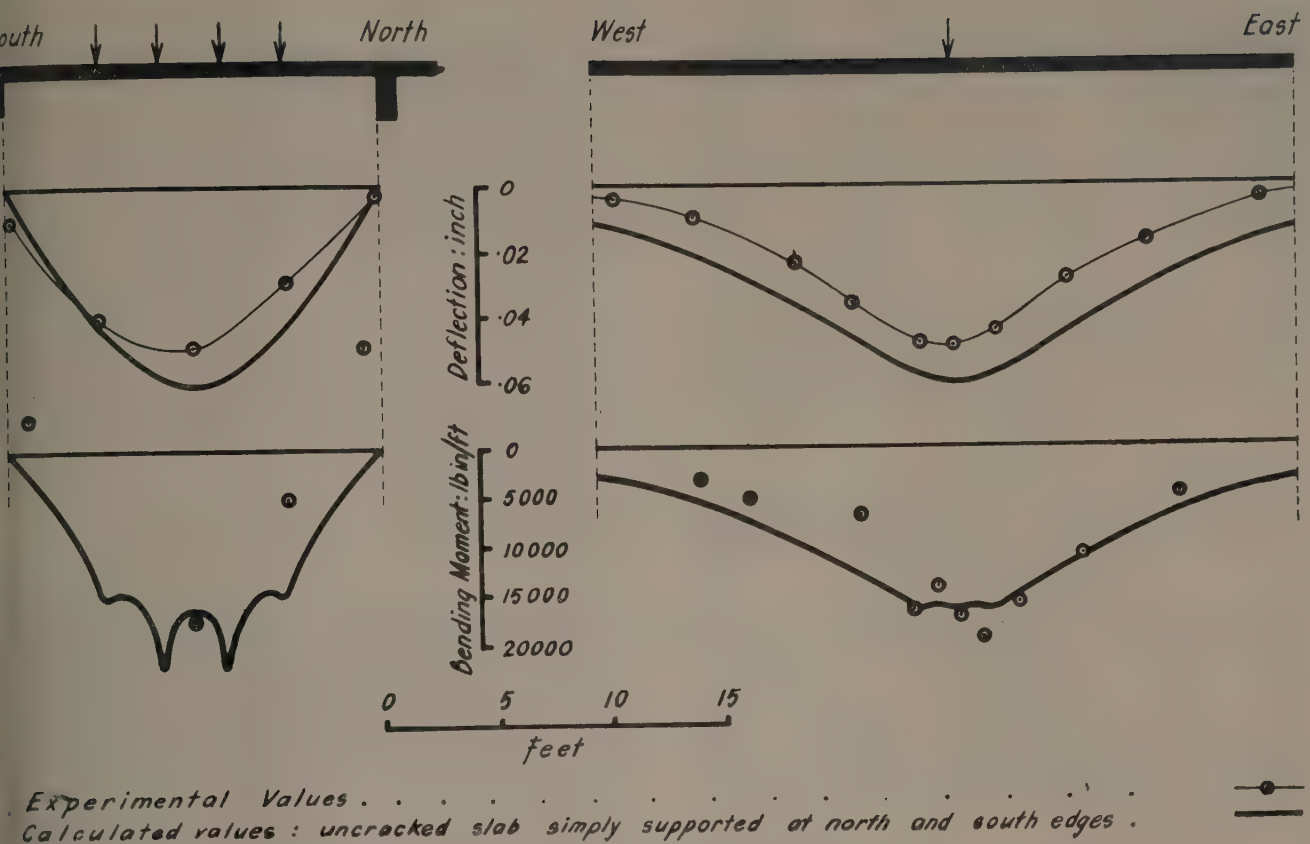


Fig. 12.—One-Way Slab, Line Loading : Deflections and Moments in Direction of Span on Transverse and Longitudinal Centre-Lines for a Total Load of 10,000 lb.

deflections and strains generally remained less than the calculated values corresponding to fully-cracked conditions (cracks extending to the neutral axis) until about three quarters of the ultimate load was reached.

In another series of tests a line of four equal concentrated loads, equally spaced from one quarter-span point to the other, was applied at the centre of a 31 ft. length of the slab, free at both ends. Some of the results obtained for loads within the working range are shown in Fig. 12. No cracking was observed and the deflections were generally similar in form to, though rather less than, those calculated by the elastic plate theory for the uncracked slab simply supported along its north and south edges. The strains, particularly at the supports, were more erratic, but the bending moments derived from the strains at midspan on the assumption that the slab was uncracked agreed moderately well with the calculated values.

When the loading was increased beyond the working range a transverse crack developed on the underside of the slab beneath the line of loads at a total load of about 48,000 lb. At the same time cracking was observed over the north support at the middle of its length. A similar crack formed over the south support at about 65,000 lb., and shortly afterwards the crack on the underside of the slab bifurcated, branches inclined at about 60° on each side of the line of loads extending from beneath the most southerly load towards the adjacent support. At the maximum load applied in this test, 78,300 lb., there were signs of crushing on the underside of the slab at the north support at the centre of its length.

In the next test a transverse crack formed across the upper surface of the slab parallel to and about 7 ft. to the east of the line of loads, at about 40,000 lb. With increased loading the cracks over the supports extended and the transverse crack on the underside opened rapidly, evidently as a result of yield in the distribution steel beneath the line of loads ; this crack bifurcated beneath the most northerly load at about 65,000 lb. At a total load of nearly 96,000 lb., one corner of the bearing plate under the most northerly load began to punch through the slab, and as a result the equalizing beam through which the load was applied tilted, and the loading rig collapsed.

When the test was repeated a transverse crack developed on the top of the slab about 9 ft. to the west of the loads at about 65,000 lb., and as the loading was increased further cracking developed, particularly on the bottom of the slab, and the concrete on the underside crushed at both supports. Failure occurred at a total load of 98,000 lb. ; the bearing plate under the most northerly load punched through the slab, but the extensive cracking (Fig. 13) and the large permanent deflections (Fig. 14) indicated that failure due to bending had been imminent. The failing load corresponded, when self weight was taken into account, to about five and a half times the total working load.

The line-of-fracture or yield-line theory[3] gave an ultimate load for bending failure which was only 60 per cent of the load at which the slab actually failed, and a mode of failure (*a*, Figs. 14 and 15) which did not agree well with the test results. In particular, the theory indicated that transverse lines of fracture due to hogging moments would develop across the slab about 3 ft. on each side of the loads, but these fractures were actually about 8 ft. from the loads. It seems that this can be attributed to the effect of the tensile strength of the concrete, which is usually neglected. Since the

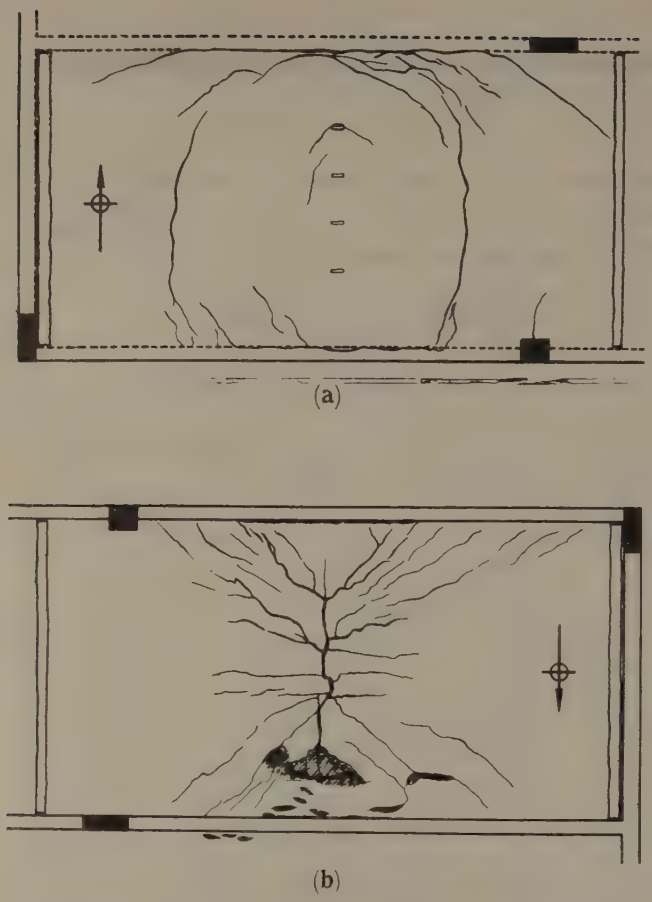


Fig. 13.—One-Way Slab, Line Loading : Cracking on (top) the Upper Surface, and (bottom) the Underside After Test to Failure

distribution steel was light and mostly in the bottom of the slab the hogging moment necessary to cause transverse cracking on the top of the slab was considerably greater than the plastic resisting moment for hogging bending, and as a result the positions of the transverse lines of fracture were determined by the resistance to cracking and not by the plastic moment. Once the cracks formed the hogging moments in the longitudinal direction were greatly reduced, and the cracks acted in much the same way as simple supports. Consequently the mode of failure, and hence the ultimate load, were influenced by the tensile strength of the concrete. When the calculations were modified and transverse lines of fracture assumed 8 ft. on each side of the loads, a mode of failure which agreed reasonably well with the test results (*b*, Figs. 14 and 15) and an ultimate load equal to 89 per cent. of the experimental value were obtained.

Tests on Slabs Spanning in Two Directions

A beam-and-slab floor system designed for a superimposed load of 70 lb./sq. ft. occupied an area 27 ft. × 100 ft. on the east of the second floor of the building. The slab, nominally $4\frac{1}{2}$ in. thick throughout, was supported by secondary beams 7 ft. $1\frac{1}{2}$ in. on each side of the longitudinal axis of the floor, and by transverse main beams of 27 ft. clear span spaced at approximately 17 ft. centres. Two tests were made on this floor to investigate the behaviour of the 15 ft. 11 in. × 13 ft. 6 in. panels bounded by the main and secondary beams, which had been designed as slabs spanning in two directions. The dimensions and reinforcement details for these slabs are shown in Fig. 16, the thicknesses and covers given being the means of values measured at a number of points.

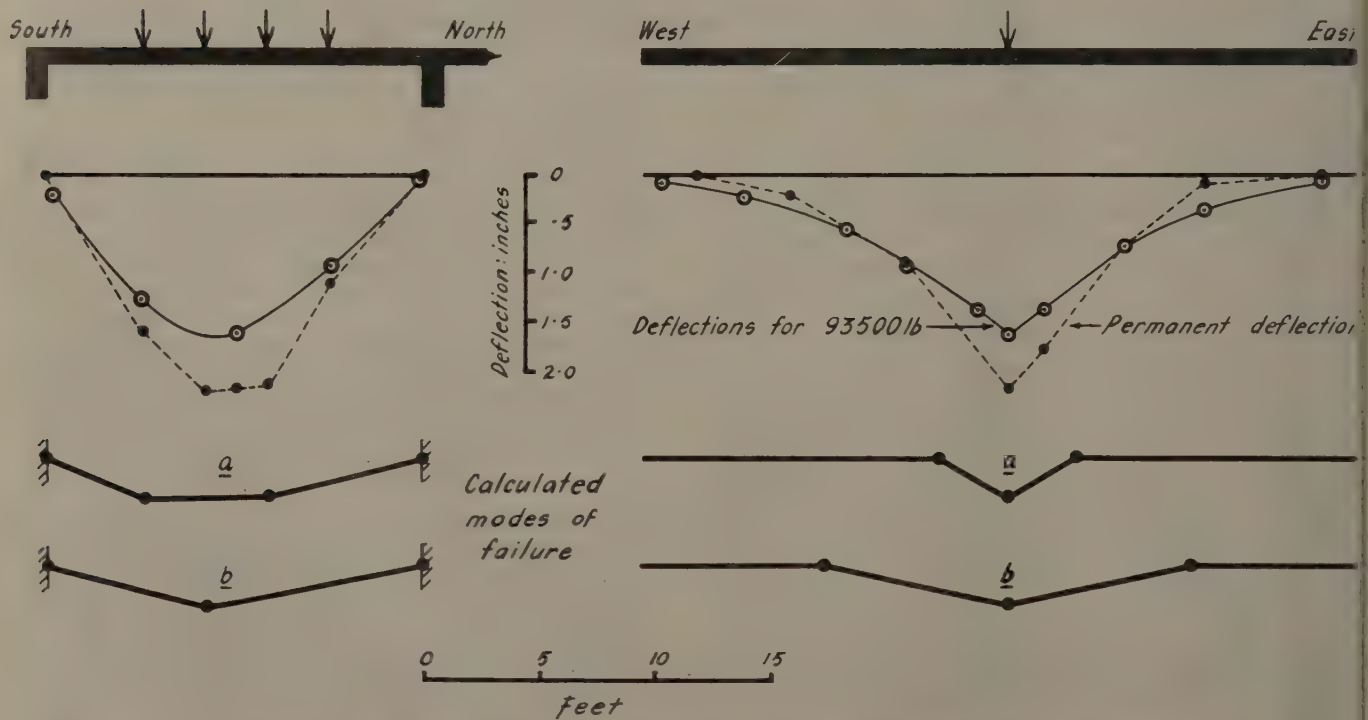


Fig. 14.—One-Way Slab, Line Loading : Deflections for 93,000 lb., Permanent Deflections After Failure at 98,000 lb., and Modes of Failure Calculated (a) in Usual Manner, and (b) by Modified Method.

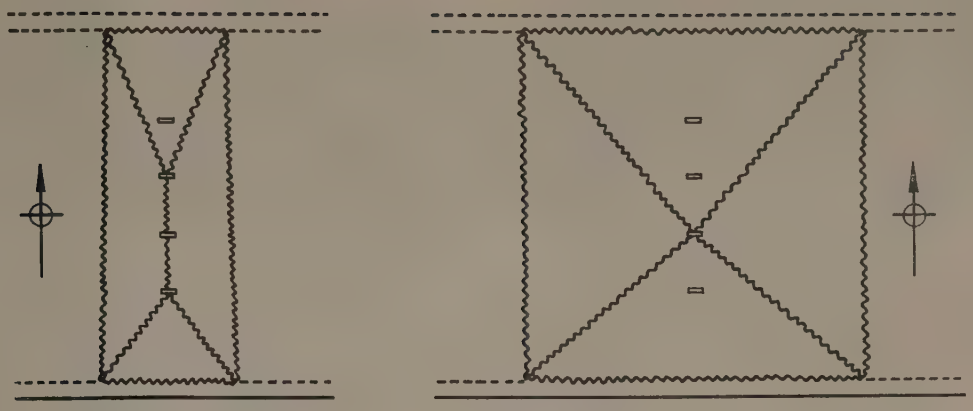
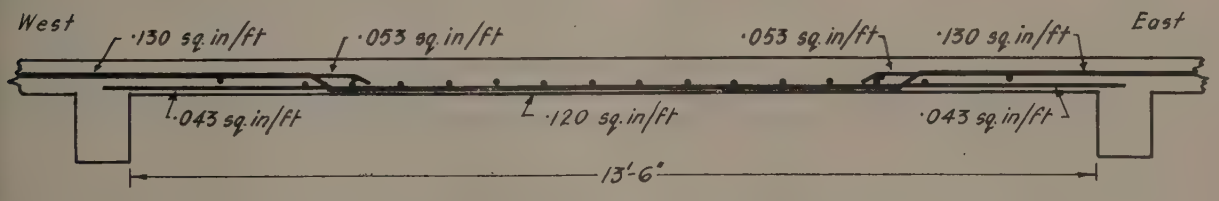
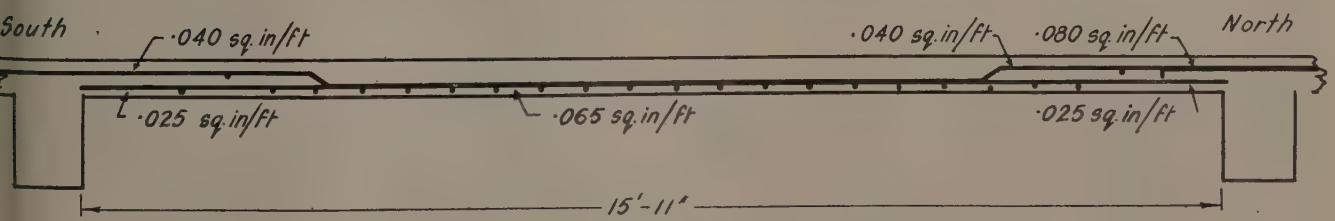


Fig. 15.—One-Way Slab, Line Loading : Modes of Failure Calculated (a) in Usual Manner, and (b) by Modified Method

In the first test uniform loads were applied to a ngle two-way panel. The load/deflection curves emained linear and the load/strain curves, though ather more irregular, approximately so until the load as about three times the superimposed load for which he slab was designed. The deflections of the beams ere appreciable, the deflections at the centres of the ain beams, the secondary beams, and the slab being the ratio of about 1 : 1.25 : 2.9. The deflections of he slab relative to the beams were rather greater than the values calculated by the elastic plate theory for the uncracked slab with simply-supported edges ; in the calculations the mortar screed was treated as an additional thickness of concrete. Nevertheless there was considerable restraint at all edges, and the moments derived from the strains on the assumption that there was no cracking were generally intermediate between those calculated for simply-supported and for fixed edges.



Section in Short Span Direction



Section in Long Span Direction

Overall thickness of slab, including ·75" thick top screed . . . . . 5·3 in

Cover over steel in short span direction : Top, including screed . . . . . 2·4 in
Bottom . . . . . ·5 in

Cover over steel in long span direction : Top, including screed . . . . . 2·2 in
Bottom . . . . . ·9 in

NOTE: Except at the main beams adjacent to end panels the top steel in the long span direction was as shown above for the north support

Fig. 16.—Dimensions and Details of Reinforcement of Two-Way Slabs

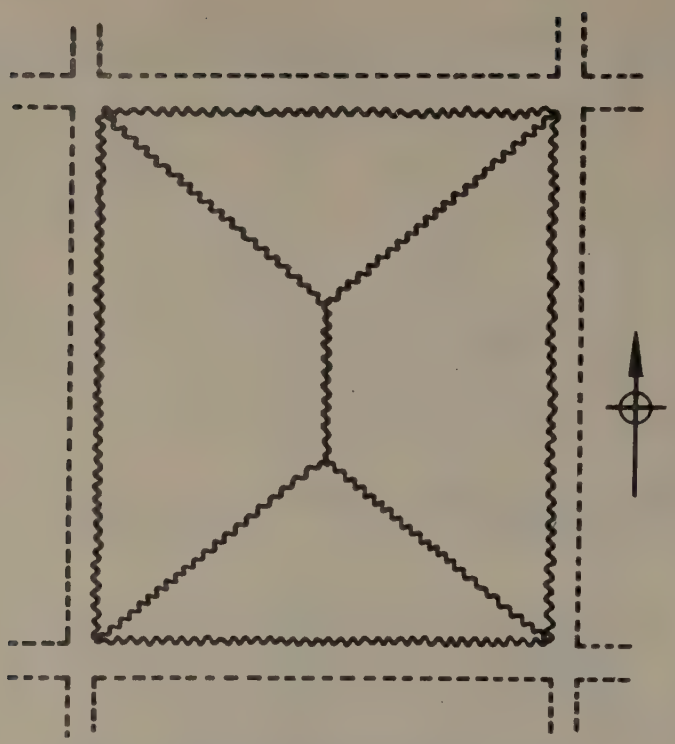


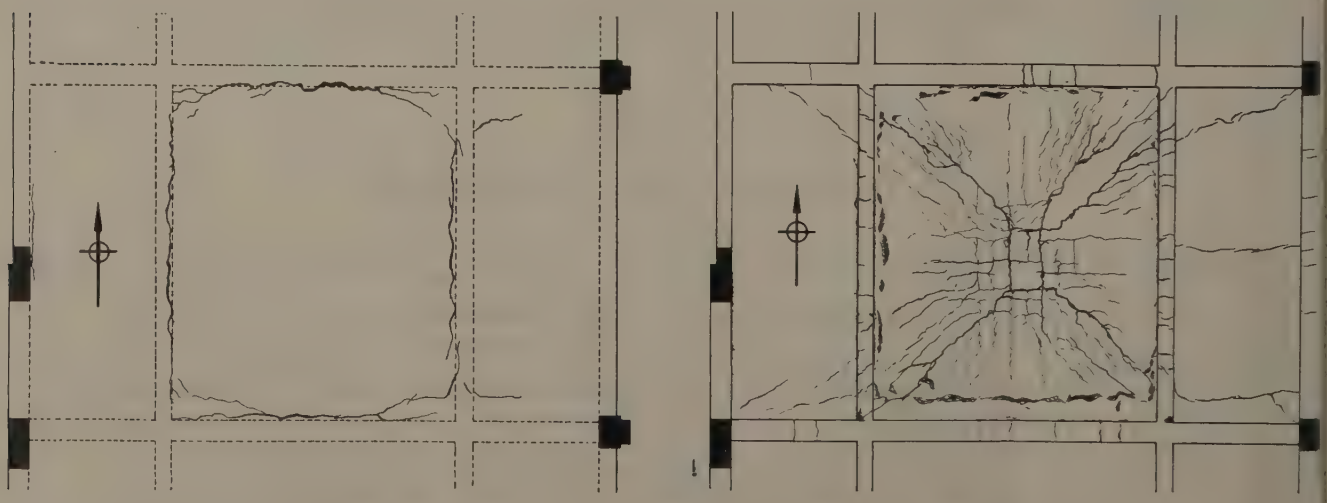
Fig. 17.—Two-Way Slab, Uniform Loading on One Panel : Calculated Mode of Failure

The first cracks formed at about 200 lb./sq. ft. on the upper surface at the centres of the short edges, probably because the main beams which supported these edges were much stiffer than the secondary beams. These cracks extended, and at about 300 lb./sq. ft. similar cracks formed over the long edges, and at about the same time cracking developed on the underside of the slab. After cracking had begun the deflections and strains increased more rapidly, but the strains did not exceed the values calculated by normal design methods from the moments for the slab with all the edges fixed until just before failure. Yield occurred in the reinforcement over the short and long edges at loads of about 420 and 500 lb./sq. ft. respectively, and the sl failed at a load of 686 lb./sq. ft. when the transver and longitudinal steel at the centre yielded and t concrete began to crush on the underside of the slab the supports. Failure was gradual, the deflectio increasing slowly but steadily under constant loa Although the negative reinforcement at the suppor was approximately an inch below the designed positi the ultimate load, 753 lb./sq. ft. including self weigh was nearly six times the total design load.

The yield-line theory gave a mode of failure (Fig. 1 which was in general agreement with the permane deflections and the cracking pattern (Fig. 18), but t predicted ultimate load was only 39 per cent. of t experimental value. This large discrepancy cannot accounted for by strain hardening of the reinforceme or by membrane action, and no satisfactory explan tion has yet been found. It seems possible that t behaviour of the slab at failure was affected by t tensile strength of the concrete. The reinforceme was very light, the maximum area of tensile steel bei only 0.2 per cent. of the gross sectional area includi screed, and the moment necessary to cause cracking the concrete exceeded the plastic resisting moment all sections.

In the second test equal uniform loads were appli to two adjacent two-way slabs. The deflections of t main beam between the loaded panels were more tha twice those of the main beams on either side (Fig. 1 but this appeared to have comparatively little effect the behaviour of the slabs. For loads within the wor ing range the deflections of the slabs relative to t supporting beams and the strains in the steel a concrete were much the same as those observed in t previous test. Until shortly before failure occurr the strains were less than the values calculated f fully-cracked conditions from the bending moments f a slab with fixed edges (Fig. 20).

Probably owing to the large deflections of the centr main beam cracking began earlier than in the previo test. The first crack, on the top of the slab at t centre of a short edge remote from the central ma beam, formed at 120 lb./sq. ft., and by the time a lo of 300 lb./sq. ft. had been applied cracks had develop over all the supports of both panels, and on the unde side of both slabs. The negative reinforcement at bo



(a) (b)

Fig. 18.—Two-Way Slab, Uniform Loading on One Panel : Cracking on (a) the Upper Surface, and (b) the Underside After Test to Failure

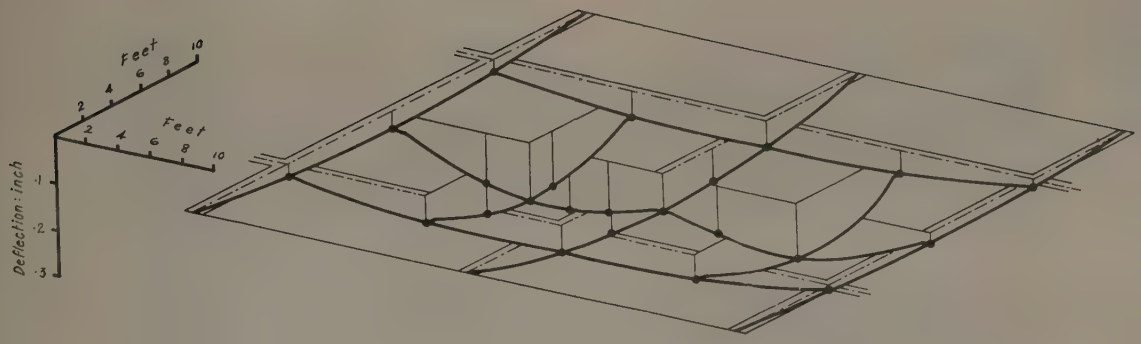


Fig. 19.—Two-Way Slabs, Uniform Loading on Adjacent Panels : Deflections for a Load of 100 lb./sq. ft.

e short edges remote from the central main beam gan to yield at about 350 lb./sq. ft., and at loads of tween 550 and 650 lb./sq. ft. yielding occurred in the eel over the other supports, in the short-span reinforcement in both slabs, and in the reinforcement at the ntre of the main beam between the loaded panels. he long-span reinforcement in both panels yielded at 6 lb./sq. ft. (843 lb./sq. ft. including self weight), six d a half times the total design load, and the deflecons of both slabs and the main beam between them creased steadily under this load (Fig. 21). The crackg and the permanent deflections of the loaded slabs ere generally similar to those for the single panel, but e cracking of the supporting beams and of the side panels adjacent to the loaded slabs was more extensive (Fig. 22), and it appeared that two complete bays of the floor system had been on the point of failure.

The main beams of the floor system were very heavily reinforced ; at midspan, where the rib was 21 in. deep overall and 12 in. wide, the tension and compression reinforcement amounted to 13.9 and 10.9 sq. in. respectively. For loads up to about 250 lb./sq. ft. on the slabs, which corresponds when dead loads are taken into account to about half the ultimate load for the beam, the deflections and strains at the centre of the main beam between the loaded panels agreed closely with the values calculated by normal design methods ; this was the only case in all the tests in

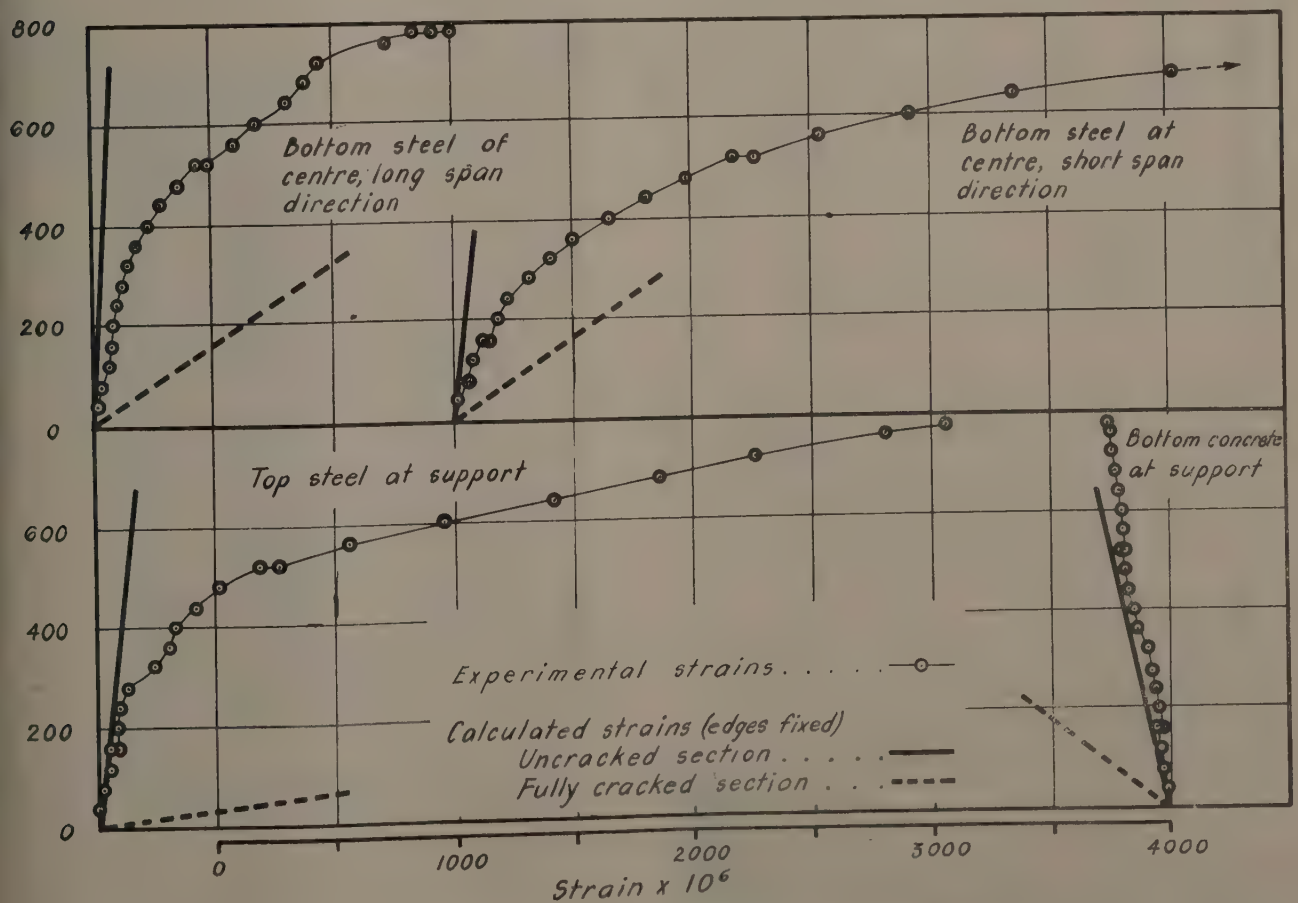


Fig. 20.—Two-Way Slabs, Uniform Loading on Adjacent Panels : Strains at Centre and at East Support of South Panel

Fig. 21.—Uniform Load of 776 lb./sq. ft. on Adjacent Two-Way Slabs

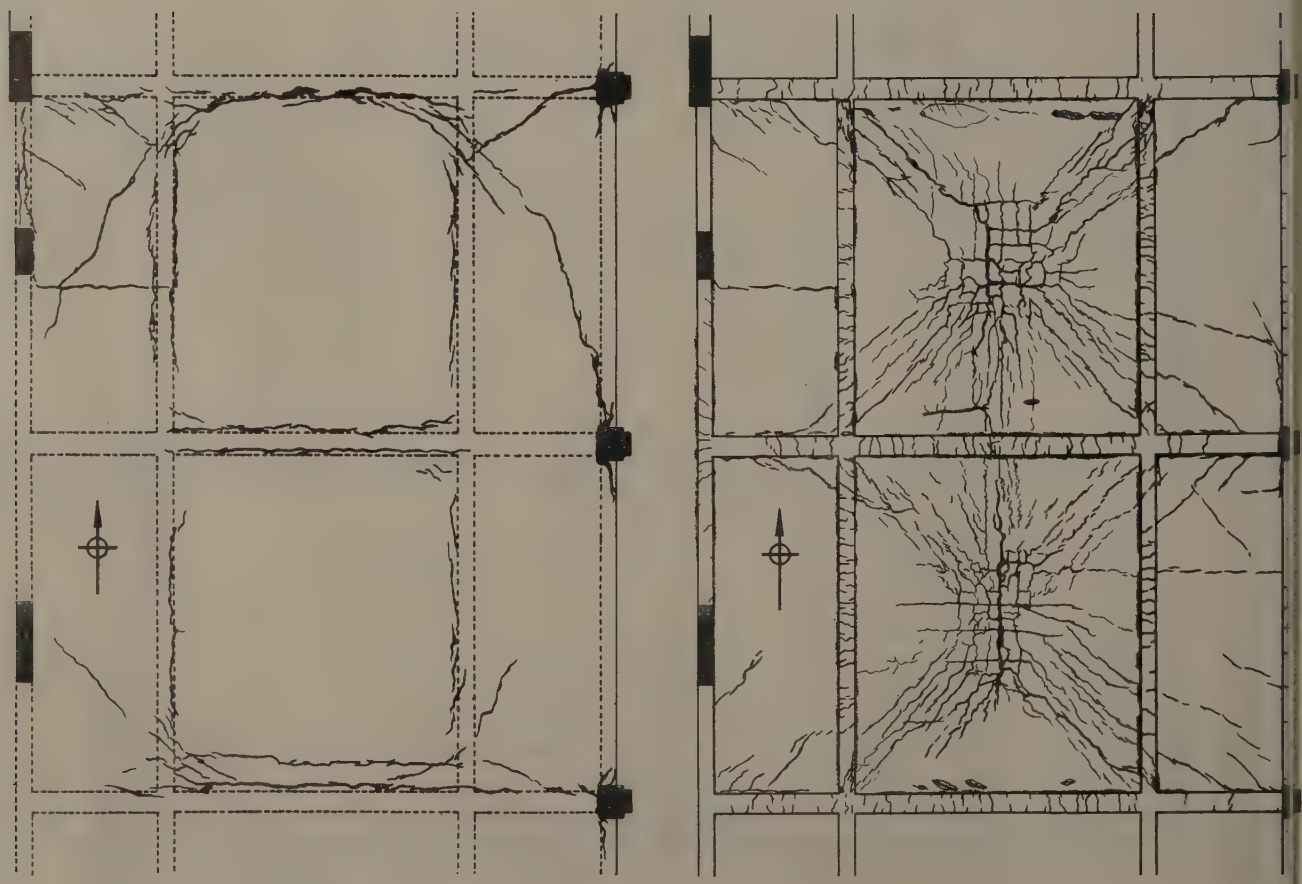


Fig. 22.—Two-Way Slabs, Uniform Loading on Adjacent Panels : Cracking on (a) the Upper Surface, and (b) the Underside After Test to Failure

hich such agreement was found. In the calculations e loads transferred from the slab to the beam and the fective breadth of the flange were estimated by the ethods given in British Standard Code of Practice P.114 (1948). The concrete on the tension side was sumed to be cracked to the neutral axis, and allow- ce was made for the restraint provided by the lumns to which the beam was connected at one end, d for that provided by the beam into which the other d framed. For the same assumptions as to the ansfer of loads and the breadth of the flange it was timated that the beam would fail, as a result of the rmation of plastic hinges at the centre and at each pport, when the load on the adjacent slabs was 0 lb./sq. ft. In view of the rather arbitrary assump- ns made in the calculations the close agreement with e actual failing load must be regarded as something a coincidence.

Tests on the Main Beams of a Floor System

Immediately below the floor on which the previous sts were made there was another of the same size and e same layout, except that the secondary beams were ft. $1\frac{1}{2}$ in. on each side of the centre-line, which had en designed for a superimposed load of 50 lb./sq. ft. ie slab was 4 in. thick throughout, and the overall mensions of the main and secondary beams were in. × 12 in. and 18 in. × 9 in. respectively; the inforcement of the main beams in the central portion the span amounted to 5.3 sq. in. In tests on the ain beams of this floor the same type of loading, a ir of equal concentrated loads applied to the main am at its junctions with the secondary beams, was ed in all cases.

The strains and deflections in the loaded main beam, ie adjacent unloaded main beams, and the slabs and condary beams between, were determined in a series tests on two of the main beams. Total loads exceed- g 60,000 lb., more than three times the superimposed ad for which the beams had been designed, were plied in these tests without causing any observable acking. The strains and deflections showed that ere was considerable bending in the slabs and condary beams in the direction normal to the span the main beam, and indicated that as a result tween a quarter and a third of the applied load was ansferred from the loaded beam to the adjacent main ams. The deflections of the loaded beam were preciably less than those calculated for the uncracked am on the assumption that there was no transfer of ad; in the calculations the usual design value was ken for the flange breadth and allowance was made r the restraint at the ends of the beam. The strains the tensile reinforcement of the loaded beam were nly a little less than the calculated values, but the rains in the concrete on the top of the rib were much naller, and corresponded to a flange breadth consider- bly greater than is usually assumed. The strains dicated that for bending in the direction of the span f the main beam the neutral axis in the slab was some- ing like 2 in. above that in the rib of the beam.

One of these main beams was later tested to failure. o cracking was observed until the total applied load as nearly 80,000 lb., when tension cracks due to sag- ing bending formed on the undersides of the secondary ams at their junctions with the loaded main beam, nd diagonal tension cracks developed in the rib of the ain beam between one of the loads and the adjacent upport. These cracks extended, and further diagonal cracking developed, as the loads were increased, and tension cracks later formed over the supports. At a load of 165,000 lb., which represents, when allowance is made for the dead loads, about four times the total design load, one of the diagonal tension cracks suddenly opened up and completely severed the concrete of the rib and the adjacent slab. Throughout the test the deflections and the bending strains remained appre- ciably less than the values calculated by normal design methods.

The maximum shear force at the section at which failure occurred was probably about 90,000 lb.; accurate assessment was difficult owing to the effect of the transverse bending of the slabs and secondary beams. The safe shearing resistance calculated on the assumption that the whole of the shear was resisted by the stirrups and bent-up bars was 16,000 lb., and the ultimate resistance corresponding to yield in the web reinforcement was 35,000 lb. A method of estimating the ultimate shearing strength proposed by Wilby,[4] in which the resistance due to the tensile strength of the concrete is added to that due to the web reinforcement, gave a value of 86,000 lb., which agrees much more closely with the test result.

The slabs and secondary beams on each side of two other main beams were cut away, and the ribs then tested as rectangular beams. For loads within the working range the load/deflection curves were nearly linear and the deflections were little greater than the values calculated for the uncracked rib. The strains, though more variable, were generally in closer agree- ment with the values for the uncracked section than with those calculated by normal design methods. When the loads were increased cracks developed at points of maximum moment at total applied loads between 45,000 and 60,000 lb., and subsequently at loads of 70,000 to 80,000 lb. diagonal tension cracks formed between the loads and the adjacent supports.

One of the beams, which framed into columns at one end and into a beam at the other, failed at a total load of 137,000 lb. The steel yielded and the concrete crushed at one of the loads, and there were signs of similar failures at each end. The failing load was more than four times the working load for the rib, and exceeded by about 25 per cent. the ultimate load calculated on the assumption that plastic hinges had formed at one load and at each support.

The other beam, which was connected to columns at both ends, failed when one of the diagonal tension cracks suddenly opened up at a load of 148,000 lb. (Fig. 23). The maximum shear force at the section where failure took place was approximately 77,000lb., nearly five times the working shear resistance for the stirrups and bent-up bars. The calculated ultimate shearing resistance, including that due to the tensile strength of the concrete, was 80,000 lb.

In these tests, as in certain of the previous tests, it was found that considerable end restraint could be developed as a result of the torsional stiffness of beams, which is usually neglected in design, and that yield stresses could be developed in reinforcement with appreciably less anchorage than design regulations would suggest.

Punching Tests on Slabs

When the main investigations had been completed tests to determine the ultimate strength of reinforced concrete slabs in punching shear were made on most of the floor panels available. The slabs tested had

Fig. 23.—Diagonal Tension Failure of Rib of Main Beam

Fig. 24.—Punching Test on Floor Slab in Progre

nominal thicknesses of 4 to 8¾ in. and there was generally a mortar screed ½ to 1¾ in. thick on top of the concrete. The spans varied from 6 ft. to over 16 ft., and the slabs were all lightly reinforced, the maximum area of steel in the tension face (the face opposite that to which the load was applied) being 0.34 per cent. of the gross sectional area. Samples of concrete from the floors tested gave compressive strengths of 3,300 to 6,550 lb./sq. in.

The slabs were loaded, usually at about midspan but in a few cases near the quarter-span point, through a steel disc 3 in. or 6 in. in diameter. In rather more than half the tests an upward load was applied to the underside of the slab by means of a calibrated hydraulic jack to which a long tubular steel strut was fitted (Fig. 24). The slab supporting the jack was generally strutted to the floor below. In the remaining tests this arrangement was reversed and downward loads applied to the upper surfaces of the slabs. Since the time available was limited only the ultimate loads and forms of failure were determined.

Failure usually occurred suddenly, a truncated cone of concrete being punched out of the slab. The small end of the cone was the same size as the loading disc; the base, though sometimes irregular, was usually approximately oval or circular in shape (Fig. 25). The average angle between the sides and the base of the cone varied for the different tests from 19° to 36°, with a mean of 28°. Failure was usually preceded by the formation on the tension side of the slab of cracks radiating from a point opposite the load. These often did not extend beyond the cone of fracture. In a few cases, however, the cracks ran right to the edge of the panel, and in addition tension cracking sometimes developed on the opposite face at the junctions of the slab and the supporting beams. When the cracking was extensive the deflections of the slab were often clearly visible before failure took place.

The failing loads for similar slabs sometimes differed considerably, but the tests indicated that the position and amount of the reinforcement, and the size of the loading disc had relatively little effect, and that the punching strength depended mainly on the thickness of the slab and the quality of the concrete. Since it had not been possible to obtain a sample from the immediate vicinity of every test the strength of the concrete was not accurately known, and the mortar screed on most of the slabs gave rise to further uncertainty. In the calculations the screed was treated as an additional thickness of concrete. This probably overestimated its effect, but the results of four tests on slabs from which the screed had been chipped away suggested that the error was not likely to be large.

Various methods proposed for the estimation of the punching strength of slabs were applied to the test results. Methods in which the punching strength is assumed to be proportional to the perimeter of the loaded area, or in which the effective depth or the moment arm is taken as a measure of the slab thickness, gave poor correlation with the experimental results. A more satisfactory procedure appeared to be that suggested by Forssell and Holmberg[5] in which it is assumed that the punching strength is determined by the maximum diagonal tension stress on the surface of the cone of rupture, given by

$$v = \frac{1.5\ P}{\pi\ (a + t\ cot\ \theta)t}$$

in which

v = the maximum diagonal tension stress,
P = the ultimate punching strength of the slab,
a = the diameter of the loaded area,
t = the overall thickness of the slab,
θ = the angle at the base of the cone of rupture.

The diagonal tension stresses calculated from the test results for a 28° cone angle (the mean experimental value) ranged from 135 to 320 lb./sq. in. For the thirty-three tests in which a 3 in. diameter loading disc was used the ratio of the diagonal tension stress to the compressive strength of the concrete had a mean of 0.5 and a coefficient of variation of 24 per cent. The corresponding values for eight tests in which the load was applied over an area 6 in. in diameter were 0.044 and 15 per cent. The difference in the results for 3 in.

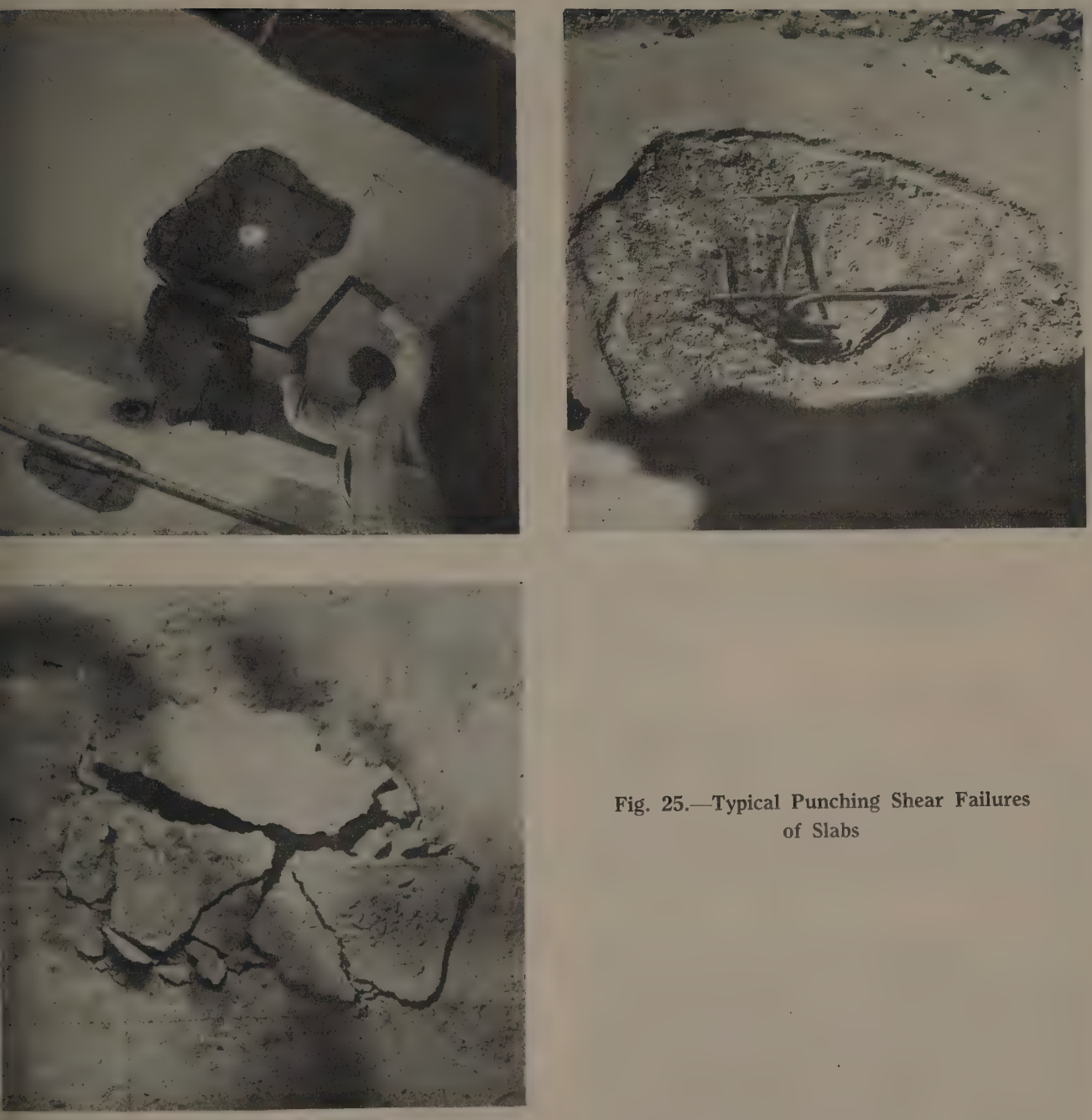


Fig. 25.—Typical Punching Shear Failures of Slabs

6 in. discs is probably not of much significance in view of the uncertainties in the data.

It has been suggested[6] that the punching strength of a slab is a function of the ratio of its punching strength to its flexural strength as computed by the yield-line method, as well as of the strength of the concrete. This theory, which appears to give satisfactory results for fairly heavily reinforced slabs, greatly underestimated the punching strength of most of the slabs tested. These were lightly reinforced, and in all except four cases the calculated flexural strength was less than the actual punching strength.

Conclusions

When the tests on the Old Dental Hospital were carried out the building had been in service for nearly ten years. The quality of the concrete in the building varied considerably, and the workmanship was by no means perfect. The defects inevitable in a practical structure, however, appeared to have little adverse effect on its behaviour.

For loads within the working range the deflections and strains were almost always less than those estimated by normal design methods, and agreed more closely with the values calculated on the assumption that the concrete was uncracked. The deflections and strains usually remained less than those calculated by conventional methods until shortly before failure. The observed deflections were more consistent than the strains which sometimes, probably owing to local cracking or honeycombing of the concrete, seemed to bear little direct relationship to the calculated values.

The ultimate loads for flexural failures exceeded those predicted both by normal design methods and by plastic theories, but were in closer agreement with the latter. The plastic theories gave ultimate loads up to about 20 per cent. less than the experimental values when failure was due to bending in one direction, but underestimated the strength of slabs failing as a result of bending in two directions by a much greater margin. These slabs were lightly reinforced, and it seems

possible that the tensile strength of the concrete influenced their behaviour.

The ultimate strength of beams which failed as a result of diagonal tension was much higher than normal design methods would suggest and the usual requirement that the shear be resisted by the web reinforcement alone appears to be unduly conservative.

Acknowledgments

This paper is presented with the approval of the Management Committee which was responsible for the organization of the tests on the Old Dental Hospital, a scheme which originated from a proposal by Mr. R. G. Ventress, B.Sc.(Eng.). Thanks are due to all those whose support and co-operation contributed to the success of the tests.

The Author wishes to express his appreciation of the work of his assistant, Mr. I. R. Scott, B.Sc.(Eng.), whose appointment was made possible by a grant from the South African Council for Scientific and Industrial Research, and who carried out the tests on the materials and many of the calculations involved in the analysis of the test results.

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Discussion Evening on Ultrasonic Testing of Concrete*

Summary of Introductory Remarks by Dr. R. Jones, of the Road Research Laboratory

The testing of concrete by the ultrasonic pulse technique involves measuring the time taken for a pulse of vibrations to pass through a known length of concrete. The path length divided by the time of propagation gives the pulse velocity, a quantity which is used as a criterion for studying the quality of the material.

Results of experimental investigations have shown that there is a correlation between the pulse velocity and the strength of a particular concrete mix. Such a correlation can be anticipated theoretically, since, in hardened concrete, the changes in void content influence the velocity and strength in a qualitatively similar manner. Changes which occur in the concrete during hardening also similarly affect both the pulse velocity and the strength. However, the relation between pulse velocity and strength is not unique, but is affected by certain factors, such as the mix proportions and the type of aggregate used in the concrete. For this reason, interpretation of the pulse velocity in terms of absolute strength requires a knowledge of the concrete mix.

When tests are made on structures in course of construction, specimens of the concrete employed in the mix are usually available. By measuring the pulse velocity through them before making the normal compressive or flexural tests, a curve relating velocity to strength is obtained for the material in the structure. Measurements of the velocity in the structure can then be interpreted in terms of strength. When older structures have to be tested for which specimens are not available, a rough estimate of strength can be obtained from calibration curves made elsewhere if it is at all possible to identify the concrete by chemical analysis or other means. In such a case, it is obvious advantageous to cut a core or some other specim from the structure to make a check on the relatic between velocity and strength. On all structures, t variation of the velocity from point to point can used to detect and locate any areas of weakness due deterioration or poor compaction.

There are now a number of published articles illu trating how the method has been successfully employ to test the concrete in dams, walls, precast product etc. The accuracy of the estimate of strength in su cases is difficult to judge, but, where checks have be possible, there was an underestimate not exceedi 15 per cent. This is probably because, on the occasions, no allowance was made for the difference moisture content between the structure and the w test cubes used to obtain the pulse velocity/streng relation. The form of this relation is such that t technique is most accurate for assessing the streng of weak concrete, and, more particularly, for studyi the variability of the concrete in a structure.

The technique is more difficult to apply when on one side of the work is accessible, as occurs with concrete road slab. A reflection technique has be developed, but it can only be applied when the unde side of the slab is relatively smooth.

The pulse method has also been used to study t hardening of concrete, its deterioration due to fro action, and to investigate the onset of cracking specimens subjected to compressive and tensile stresse In the latter investigation, it was found that, in cub in compression, initial cracking occurred parallel to t direction of application of the load at 30-70 per cen of the ultimate. The start of the internal fractu depended upon factors such as the uniformity of t stress distribution and the type of the coarse aggregat

**To be held at the Institution of Structural Engineers, 11 Upper Belgrave Street, London, S.W.1, on Thursday, 27th October, 1955, at 6 p.m. The discussion will be preceded by a demonstration of the apparatus.*

Structural Prestressing*

Discussion on the Paper by Mr. F. J. Samuely, B.Sc.(Eng.) Lond.), M.I.C.E., M.I.Struct.E.

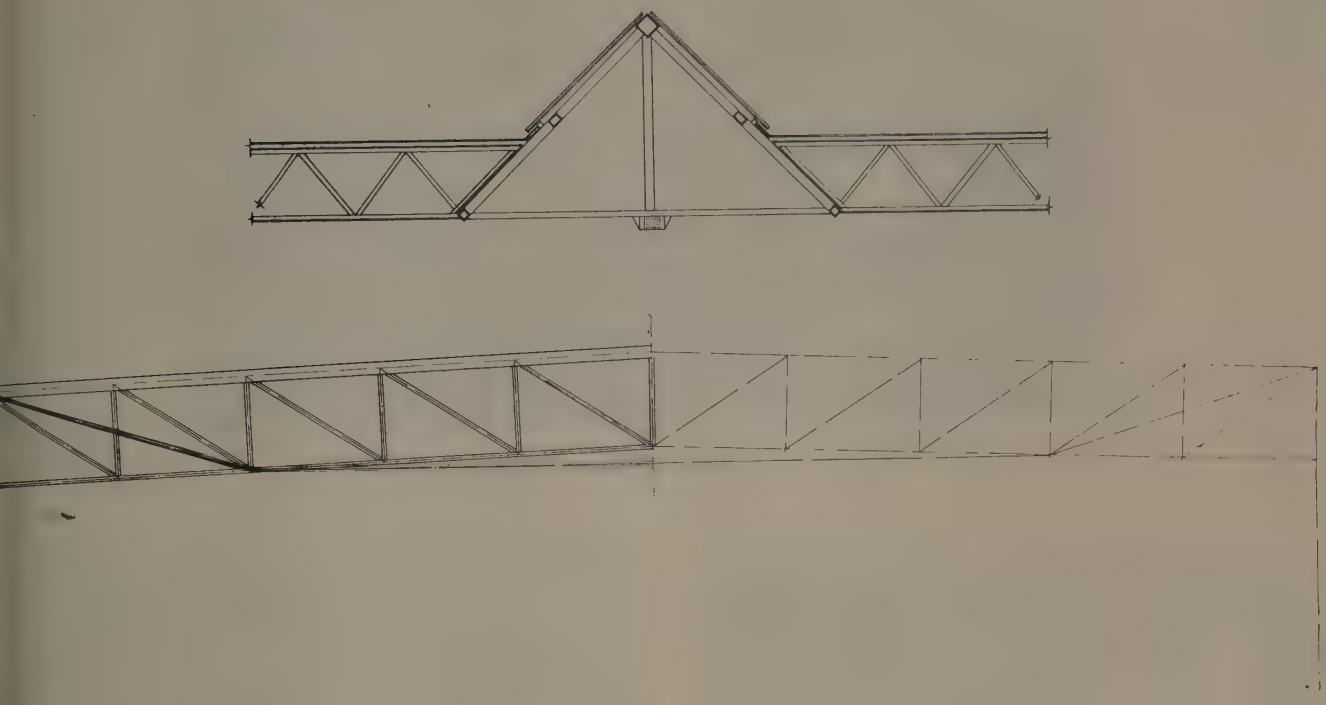


Fig. 1

THE CHAIRMAN introduced the Author, who then presented his paper.

THE CHAIRMAN proposed a vote of thanks to Mr. Samuely for his interesting paper and for the very pleasant way in which he had put over the main substance of it in the abstract which he had just given.

(The vote of thanks was carried with acclamation.)

Discussion

Mr. R. A. SEFTON JENKINS (Associate-Member) said that he had thoroughly enjoyed reading the paper and had been most amused at the idea of the Romans prestressing their abutments to arches by adding load over the top of the abutment. It was most ingenious of Romans to have done this, and also of Mr. Samuely to have thought of it.

In his paper the Author had said that prestressing, as such, normally reduced deflection. Mr. Sefton Jenkins felt that this was not true in most cases. Taking a prestressed structure such as had been shown in Fig. 2—a pitched roof space frame, prestressed by wires or cables in the form approximating to a parabola —the deflection would be exactly the same under a given load whether the wires were tensioned or not. This was, of course, assuming that the structure and the area of the wires were the same in each case.

Mr. Sefton Jenkins showed illustrations of two jobs on which he had recently been working.

Fig. 1 showed a structure rather similar to Mr. Samuely's last example. It was a space frame of about 100 ft. span with glazing on the two sloping sides and with prestressing wires on the centre line of the girder. The wires started at the top of the girder at the ends, came down to the level of the bottom chord and then ran horizontally to the other side. As the girder was cambered along its length, the wires came below the bottom chord at the centre of the span. The beams were at 50 ft. centres and had lattice beams between them supporting flat roofing.

Figs. 2, 3 and 4 showed a roof of 80 ft. span consisting of prestressed R.S.J. This was a rather interesting structure because it was a portal structure but only the beam was prestressed. For various reasons it was desired to keep the bending on the foundation to a minimum and to introduce pin joints at the bases of

*Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1 on the 10th February, 1955, Dr. S. B. Hamilton, M.Sc.(Eng.), A.R.C.S., M.I.C.E. (President) in the Chair. Published in THE STRUCTURAL ENGINEER, Vol. XXXIII, No. 2, pp. 41-54, February, 1955.

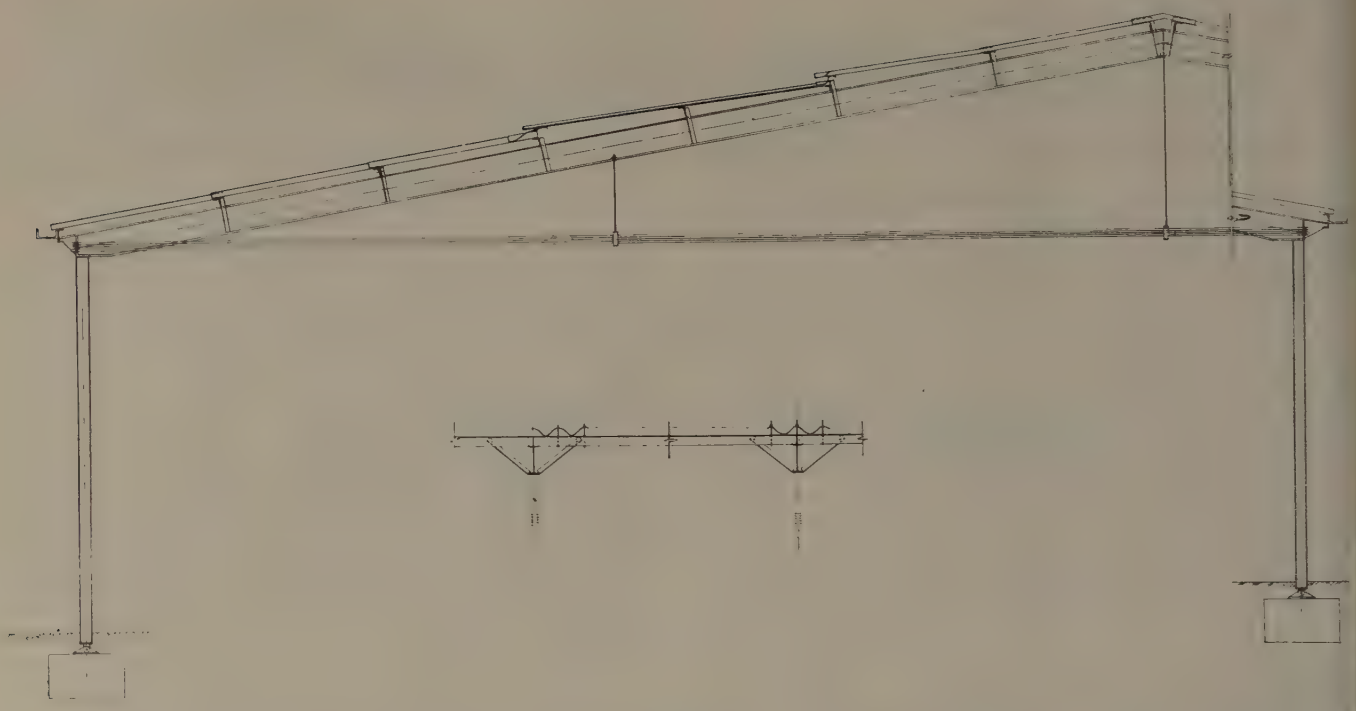


Fig. 2

columns. This necessitated the moment due to wind being taken by the beam. If the whole portal had been assembled, welded up, and then prestressed, very large moments would have been induced into the columns by the shortening of the beams by prestressing. The method of erection laid down was therefore that firstly the columns should be erected and stayed and the beams placed on the stanchions, secondly purlins etc. were added, and lastly the beams were then prestressed. The joints between the columns and beams were then welded up and the structure completed.

The wires used for prestressing were high tensile wires of 0.276 in. diameter, such as are used for prestressing concrete, except that they were galvanised. Fig. 4 shows a pair of Gifford Udall jacks being used to stress one of the girders.

Mr. SAMUELY, in reply, said that, as he had already stated, there was normally a downward deflection, for instance, in a beam due to dead load or part of the dead load ; prestressing then produced an upward deflection. A downward deflection was then caused by the remainder of the dead load, or the live load—or in Mr. Sefton Jenkins' example, by the first part of the load —resulting in a final downward deflection. In other words, the various deflections went alternately down and up, and under equal circumstances the final deflection is less than the normal deflection in a structure that is not prestressed. As the upward deflection caused by the prestressing works against the downward deflection caused by the load, with sufficient prestress it is possible to make even the final deflection upward.

On the other hand however, high tensile steel is very often used for structures of this type, which has a greater deformation than material of lower stress capacity. Quite obviously a cable that can be stressed up to 35 or 40 tons per square inch must have a greater deformation than steel which can take only 9 or 10 tons per square inch.

These two factors balance each other to a certai extent. Using high tensile steel to get the most ecor omical structure might not show any advantage as fa as the deflection was concerned. To improve on th deflection a compromise might be necessary, whic would probably still be more economical than mil steel. Full advantage could not be taken of bot factors, but it was usually possible to have one or th other, or a little of both.

Mr. A. GOLDSTEIN (Associate-Member) said M Samuely had given a readable and erudite account c prestressing.

The paper considered prestressing in a fundamenta way, and when Mr. Samuely started his introductio he had given a definition of prestressing which migh perhaps, be put in another way. A definition of pre stressing which he personally liked was that pre stressing meant the introduction into a structure c artificial permanent stresses of such a nature as t eliminate or reduce those naturally occurring stresse which the structure was ill-conditioned to resist. Th only main difference between the definition given b the Author and that one was the use of the terr " artificial " as against " preliminary " which M Samuely had used. On page 42 the Author had use the word " artificial " and from that one might believ that on the whole he favoured it. If that were so, h did not think that the arch shown in Fig. 1 fell withi the scope of the definition ; neither did gravity struc tures of any sort. In his view the term " artificial was important because it separated those " mar made " stresses from the naturally occurring stresse and it was important because the man-made stresse were under the control of the designer whereas th naturally occurring ones were not. Again, th artificially-induced stresses were subject to certai variations, problems such as creep, and so on, and th natural stresses were not. The distinction was a

Fig. 3.—80 ft. span prestressed steel roof

mportant one when the whole problem was considered. The naturally occurring stresses were not normally ubject to variation.

In the same way the silk of an umbrella was not an ipt illustration of prestressing. It was a good illustration of a stressed-skin structure but it did not apply o prestressing, as such, in a particularly apposite way.

In his view the clearest way to illustrate prestressing was not by a consideration of indeterminate structures : after all, one liked to keep to simple structures wherever possible and prestressing could be explained quite satisfactorily by using simple structures and without recourse to indeterminate ones.

There was a statement on page 43 : " It will be noted that this reduction in deformation is a general characteristic of prestressing." Mr. Goldstein thought that this statement, taken as a generality, was misleading. One had first of all to define what was meant by " deformation." If one referred to strains then, of course, prestressed structures, having larger stresses as a rule—and he was talking about generalities—had larger strains, but Mr. Samuely had probably referred to deflections. In connection with deflections he agreed that the own weight deflections of a prestressed structure were generally less than those of a non-prestressed structure, and again if one took a beam, say, 10 in. wide and 20 in. deep in reinforced concrete and one took the same cross-section in prestressed concrete, the deformation of the prestressed concrete beam would be less than the reinforced concrete beam ; but the important thing to remember was that those cases were hardly ever comparable. A beam was designed to take a certain load, and due to prestressing, the resulting cross-section of a prestressed concrete beam for a particular load was generally smaller than the equivalent reinforced concrete beam. The own weight deflection would be less, sometimes negative, but the deflection for applied load would be considerably greater and hence the prestressed concrete beam would usually be more flexible. It was an important distinction because we were not as a rule—and speaking generally—concerned with own weight deformation : allowances for those could be made in the construction. Those deformations only occurred once in the life of the structure, and provided always that they were not so great as to introduce dangerous secondary stresses, they were not of prime importance. But one had to be very much concerned with live load deformation, and that had become one of the problems demanding a great deal of attention because it was being found that with prestressed construction members were becoming so small in cross-section that in addition to consideration of stress and strain they also had to be studied from the point of view of deflection and vibration. Mr. A. D. Holland of the Ministry of Transport, in a paper which he recently presented to the Institution of Civil Engineers, mentioned that particular point in connection with small span highway decks, and he showed that for fairly small spans one of the criteria

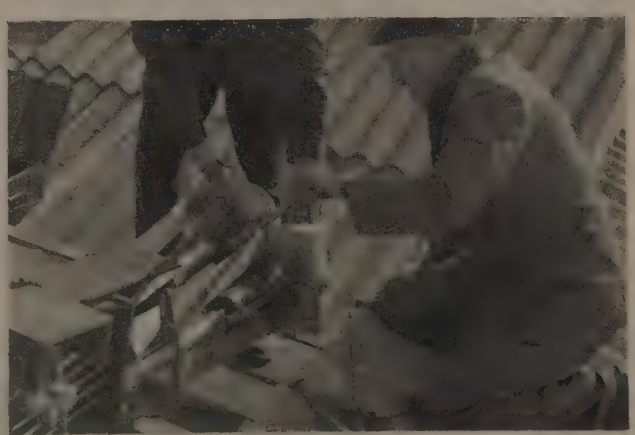


Fig. 4.—Two Gifford Udall Jacks being used together to prestress 80 ft. span roof

for prestressed concrete construction was the natural frequency of the deck ; due to the small sections which had become possible the condition could arise where the natural frequency was so low as possibly to permit induced resonance due to normal live loading, and that, of course, was a serious state of affairs. Mr. Samuely had quite rightly pointed out that for large suspension bridges where obviously their own weight was the main consideration, the principle of prestressing could be used with advantage to reduce deflections.

Concluding his remarks Mr. Goldstein said that the points which he had made were points of detail and Mr. Samuely would undoubtedly have his answers for them. It was a very well written paper—some might think that in places it was perhaps a little laborious—but it was quite clear that the Author had taken very great pains in presenting the subject and therefore deserved members' thanks and appreciation.

Mr. SAMUELY, in reply, said Mr. Goldstein's contribution had been very interesting. He was in complete agreement with the definition put forward, and in his paper when he talked of preliminary stresses he really meant that the stresses were put in at an early stage. However, while he agreed to the definition of prestressing as being the introduction of artificial stresses he was quite sure that the stresses in the Roman arches were artificial because there was absolutely no need to put a load at that particular point, nor to express it architecturally. The load was not necessary and the arch, as such, would have been exactly the same without the load on the pier. It was quite definitely done deliberately : everything above the arches was left hollow and just on top of the pier a very considerable load was introduced where there would have been quite a good saving, even for Roman considerations, if that load had not been introduced, so that he felt quite justified in calling that loading artificial.

With regard to the question of the umbrella, he quite appreciated it was certainly not the purpose of an umbrella to be stressed and in that sense it did not come under prestressing, but at the same time it was really meant only as an interesting example of what could be achieved if one wanted to prestress. It did not really matter whether the prestressing was accidental or deliberate, the fact remained that the silk, due to the stress which was introduced, became suddenly very rigid indeed.

On the question of statically indeterminates he remained quite firm. He did not think Mr. Goldstein could show something that was statically determinate and that could be prestressed, but in order to make it quite clear he would not call an ordinary lattice girder with a cable statically determinate because the cable made it statically indeterminate, and a reinforced concrete beam was always indeterminate because it would be determinate only if there was no reinforcement in it. However small the capacity, that concrete beam was a determinate structure and tensile steel was welded to it which made it indeterminate. It was calculated in that way by applying the formula which took the relating strain into account. He had always understood that statically determinate structures could be calculated by equations of equilibrium, and stresses were defined and could not be altered except, of course, by application of external loads.

With regard to the deformation, he had not assumed in his paper that a prestressed structure was necessarily shallower than a non-prestressed structure : if it were shallower then he quite agreed the deformations were larger, but it should be appreciated that very frequently prestressing was used because the material was not capable of fulfilling a particular requirement. For instance, ordinary concrete was not capable of taking tensile stress ; it was liable to crack and as such it would deflect more than a prestressed structure under equal circumstances—therefore prestressing structure meant, in many cases, a reduction of deflection. However, he agreed, undoubtedly if anybody tried to go to the limit of what a prestressed structure could do, this would lead to trouble : with too shallow a construction either the deflection would be too great or the frequency too low, either of which could be dangerous.

Mr. H. C. VISVESVARAYA said that he wished to join the previous speakers in congratulating Mr. Samuely on his excellent paper and to thank him for having made available some useful information regarding both prestressed concrete and prestressed steel, about which very little was known at the moment.

With regard to the method of prestressing, Mr. Visvesvaraya said that it would be interesting and useful to review the fundamentals of prestressed concrete before taking a decision on any particular method of inducing the prestress. Although he proposed to confine his remarks to prestressed concrete, the general principles were applicable to prestressed steel also. He, therefore, considered two simply supported concrete beams, each beam being made in two halves and joined by prestressing as shown in Fig. 5. The prestressing in the case of Beam "A" was by direct jacking and subsequent wedging, and that in the case of Beam "B" was by the use of some sort of normal tendon (such as, say, H.T.S. Wire). It was assumed that both the beams were identical in size, quality of concrete, span etc., and that the same amount of initial prestress was put in both. If one studied the behaviour of both the beams under identical loading, one would observe

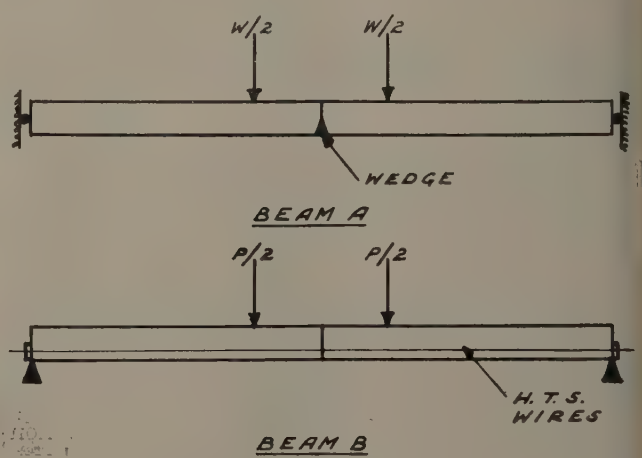


Fig. 5

the behaviour of both to be almost identical until the cracked. Theoretically, cracking of both the beams occurred simultaneously and at the same load because the initial prestress (neglecting creep of steel, or abutment movement of creep etc.) was the same and both the critical sections were midspan joints. But immediately the beam cracked there was a transformation and that subsequent to cracking Beam "A" was no more than two pieces of concrete beams, the wedge having slipped out or even if kept in position being

ere without doing any work, and the beam would fail mediately. On the other hand in the case of Beam B," immediately the beam cracked the steel wire or r came into action and hence Beam " B " would ntinue to take considerably increased loads in the me way as an ordinary reinforced concrete beam but th prestrain in the steel. Therefore, the beam with-t any reinforcement would fail immediately after it acked without giving any warning, whereas the beam th the reinforcement would continue to take very uch higher loads and if the beam was under-reinforced gave sufficient warning before failure. It was, there-re, obvious that in order to obtain the same ultimate ad carrying capacity a given beam would have to ve a far higher initial prestress when the method of estressing was by direct jacking and or wedging than hen it was by the prestraining of the normal tendons. he former method was naturally an uneconomical roposition not only from the point of view of the cost inducing those high prestresses but also from the oint of view of the losses which were in a way pro-ortional to the initial prestress. Lack of signs of arning before failure was a further serious drawback. was for these reasons that Mr. Visvesvaraya felt that he method of prestressing by direct jacking and/or edging was not at all a recommendable proposition.

A similar argument was also true in the case of the ridge shown with the slide by Mr. Samuely. While he bridge had been made to act as a rigid structure y using the Freyssinet flat jacks at the abutments, nd while it was true that the bridge behaved like a gid structure as far as one could normally see, the ue value of the bridge in terms of its ULTIMATE OAD CARRYING CAPACITY was, in his opinion, o more than that in which there was no rigidity at the butments because of the fact that when once the crack ppeared the effect of rigidity was lost.

In the same connection he referred to Fig. 7 in the aper which was a similar case, and said that such a tructure would be either uneconomical or dangerous.

Considering next the question of continuous beams, e referred to Figs. 4a and 5b, and said that similar chemes were considered by him at Imperial College ut they were abandoned as the advantages of such chemes did not, in their opinion, compensate the dis-dvantages.

Going into some details Mr. Visvesvaraya said that e presumed that Mr. Samuely had suggested to place he jacks at the zones of contraflexure as otherwise it nay become very difficult to deal with the local stresses. With regard to the turnbuckles such as the one shown n Fig. 8, Mr. Samuely had mentioned that it would ntroduce a moment at the support opposite to that lue to the superimposed load. While the primary prestress moment would be of opposite sign, the econdary moment (such as that shown in Fig. 4b) would be of the same sign as that due to the super-mposed load, and since this secondary moment depended not only on the primary moment but also on the length over which it extended, i.e. the length of the cable, great care was needed in the design. He said that in some cases the secondary moment may be greater than the primary moment, thus completely nullifying the primary moment. A simple example was the case when a straight cable along the whole length of a two-equi-span continuous beam was stressed the secondary moment was 1.5 times the primary moment. Mr. Visvesvaraya asked the Author whether he could give some details of operation of the turn-buckles because he had found that beyond a certain load it was very difficult, nay, impossible to operate the turnbuckles ; for example, a 1 in. rod having ordinary Whitworth standard threads and nuts could not be manually operated beyond a load of 10 tons.

Mr. Visvesvaraya did not think that prestressing was in competition with plasticity as Mr. Samuely had mentioned. First of all Mr. Samuely's definition of plasticity appeared to need amendment. Secondly, all that Mr. Samuely said was true if only one was considering the structure up to its cracking stage only and not beyond. But, in practice with normal reinforcement and prestress the beam continued to carry very much increased loads till it failed and before it failed there was a plastic behaviour and redistribution of stresses.

With regard to the question whether a statically indeterminate structure could be prestressed or not, Mr. Visvesvaraya said that he agreed with both Mr. Samuely and Mr. Goldstein. The only difference that he saw was that Mr. Samuely was considering the structure during or after it was prestressed and Mr. Goldstein before it was prestressed. He therefore thought that a statement saying " a structure cannot be prestressed without its becoming an INDETERMINATE STRUCTURE after prestressing " would clarify the matter. A statically determinate beam could, therefore, be prestressed but it would become an indeterminate structure after prestressing.

Dr. H. Gottfeldt (Member), referring to the Author's definition of prestressing in which it was insisted that the laws of equilibrium must be maintained, said he always told his students that if they knew those laws all their troubles would be over : but one of them had come up to him and said, " Mr. Samuely does not seem to think so because on pages 51 and 52 point E is not in equilibrium." He had been somewhat taken aback at that and he believed it was tied up with the question of the connection of the cable which came down, and it would be interesting to have Mr. Samuely's views on that point.

Illustrating his point on the board, Dr. Gottfeldt said that if the connection was so that only a vertical force could be introduced into the vertical member, then there was something wrong with the equilibrium.

It would also be interesting to know exactly how the Author determined the optimum amount of prestress : was he certain that what he had done at the bottom of page 52 was the optimum, or could there be any other stress in the cable which would also provide something good and reasonable ? For instance, would it not be reasonable to have the whole tension in that cable ? His own view was that these cables were a new material and the most should be made of them.

Giving a further illustration, Dr. Gottfeldt said that in making a roof one could make a cranked beam, put a tie member in and prestress it, and one would then get 17 per cent. of $\frac{WL}{8}$: would it not be better if one made it a three-pin frame and just put a cable as a tie bar ? If the high tensile cables were used thus, one would then get 25 per cent of $\frac{WL}{8}$ which would be right for any type of loading whereas the other was only right for one particular type of loading. It was a sort of self-stressing structure instead of a prestressed one, and he somehow felt in spite of all the ingenious devices which only applied to specific problems in principle, an effort should be made to use the tension cables without prestressing.

Mr. F. J. SAMUELY, replying to the discussion, said Mr. Visvesvaraya's contribution had been exceedingly interesting and unfortunately there was not time to reply to it in full. In all prestressed construction the load deformation line showed a kink at some point. It was not quite correct to say that the wedges must fall out when one came to that point—there were means of keeping them in that position. (*Illustration on board.*) From the point onward when the wedges were no longer fixed, they were not active any more, exactly as in prestressed concrete—it was known that there was such a line and there was, of course, exactly the same line in ordinary reinforced concrete—and one had to have a safety factor which allowed for that. It was in the designer's hands to provide that factor of safety.

With regard to Figs. 4a and 4b, they were merely sketches and he had not gone into the question of whether or not it was advantageous to prestress. That was an entirely different matter and obviously the advantage depended very much on the span : if one had a long-span bridge it might be very advantageous although one had exactly the same sort of beam. In all probability one would not stress a 10 ft. construction whereas exactly the same sort of construction of 200 ft. in length would be prestressed. The other point, that if one had a cable which was fixed at both ends then the situation would be different, was correct. If he had such a cable he would know perfectly well the bending moment would be reversed, so he would put the cable at the bottom instead of at the top ; but it would be rather inefficient, so it was much better to have a turnbuckle which was really anchored.

With regard to Dr. Gottfeldt's remarks, unfortunately it seemed that an error had crept into the copy, for which he apologised—and Dr. Gottfeldt's student had no need to despair on the question of equilibrium ; As far as optimum strength in the cable was concerned, Dr. Gottfeldt was quite right : if one was perfectly free to do what one wanted and there were no other considerations, then it was right to take, say, the full dead load and half the live load and divide that by the depth of the beam and take that force into the cable. There were all sorts of limiting factors however, as, for instance, in the case he had quoted, where only one type of cable was available and the architect limited the number of cables which were to be used for architectural reasons.

Dr. Gottfeldt's determinate structure was certainly determinate, but it was not prestressed—one could not do that. That was another disadvantage, that if one used a high tensile cable in order to save steel, then one found the deformation was extremely large. In the case of an ordinary two-hinged frame on foundations there was very often the trouble that the ground would not be strong enough to take the horizontal force and one had to put a tie member in. The stress in the tie member would then probably be greater than the floor slab could take, and the concrete would be overstressed, and crack. The example given was perfectly all right and there was nothing to be said against it provided that the deflection did not become too large. It was a lightly stressed beam (*indicating on board*) so one just could not use it, but as far as the definition was concerned it was not prestressed so therefore it was not a question of prestressing a statically determinate system : it was an ordinary statically determina system where it was perfectly possible to use a hi grade steel.

THE PRESIDENT thanked Mr. Samuely for replyi to the discussion and invited further written cont butions.

The proceedings then terminated.

Written Contribution

Mr. H. D. OSBORN : Referring to the Autho remarks on page 43, regarding the use of tubes or oth closed sections in prestressed steel construction.

If a truss having tubular booms is to be prestresse a cable formed of the required number of wires a having spacer grilles at intervals, similar to those us in the Magnel system, and of the same diameter as t bore, may be passed through the tube and tensioned the usual way. This method would be particula suitable for exposed structures where the strength the tensioning wires may be impaired by corrosic Furthermore a tubular section is more suitable to ta the initial compressive stress, and the bottom boom o truss may be given a parabolic form.

It would be interesting to know if there are any pr stressed tubular structures in existence, and to have t Author's comments on prestressed steel structures wi small diameter tensioning wires exposed and liable corrosion.

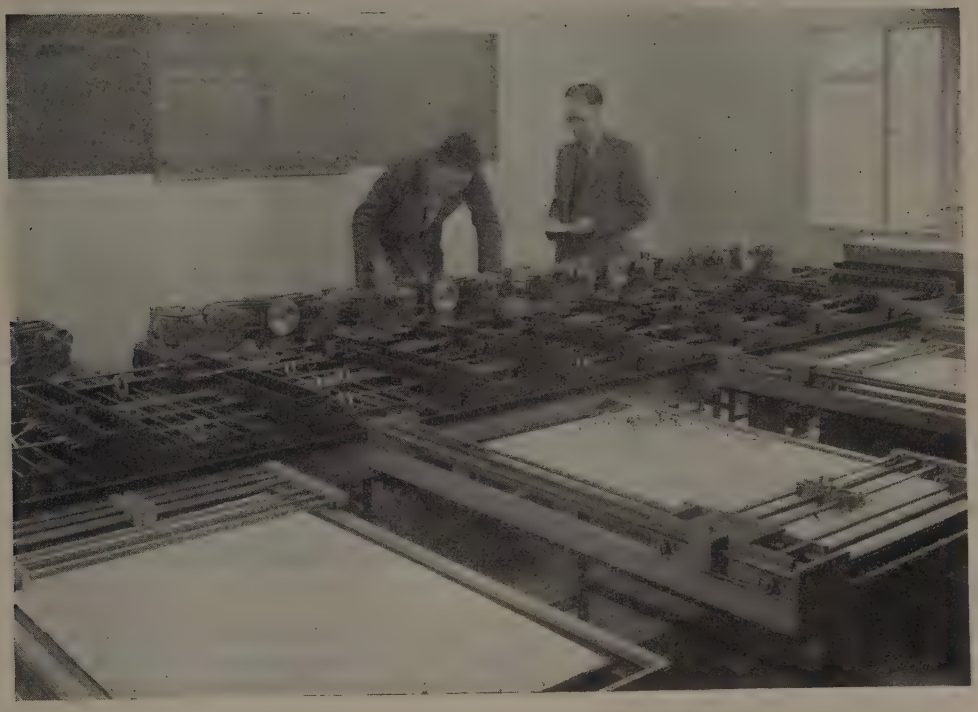
Mr. SAMUELY replied : I quite agree that the meth suggested by Mr. Osborn is possible, but I believe th it is rather costly in practice. No doubt in spec cases, particularly for exposed structures where t strength of tensioning wires may be impaired by cor sion, it will be the better answer ; but in other instanc where the danger of corrosion is not so great an op section may be preferable.

I have no knowledge of existing prestressed tubul structures. I am aware that where tensioning wi are exposed they have to be kept painted to avo corrosion.

Since giving the lecture Professor A. J. Francis the University of Melbourne has written to me a drawn my attention to several papers of his, some which have been published in the *Australian Journ of Applied Science*. These papers show that Profes Francis has worked on very similar lines to those have tried to lay down in my paper, and he has a gone in a more detailed way into the calculation prestressed structures. I would like to quote t following paragraph from his letter to me :—

" The interesting thing as regards prestressing that the approach which I have adopted leads ve simply to the conclusion that any redundant p jointed framework may be designed for a given syste of loads so that all members carry any desired str under load, provided the redundant members a inserted during construction with certain predetermin lacks of fit : that is to say provided these memb are prestressed. The drawbacks of the method present is, of course, that it applies to design for single load system. I am at present working on extension to design for several load systems."

The Building Research Station Open Days for Civil and Structural Engineers



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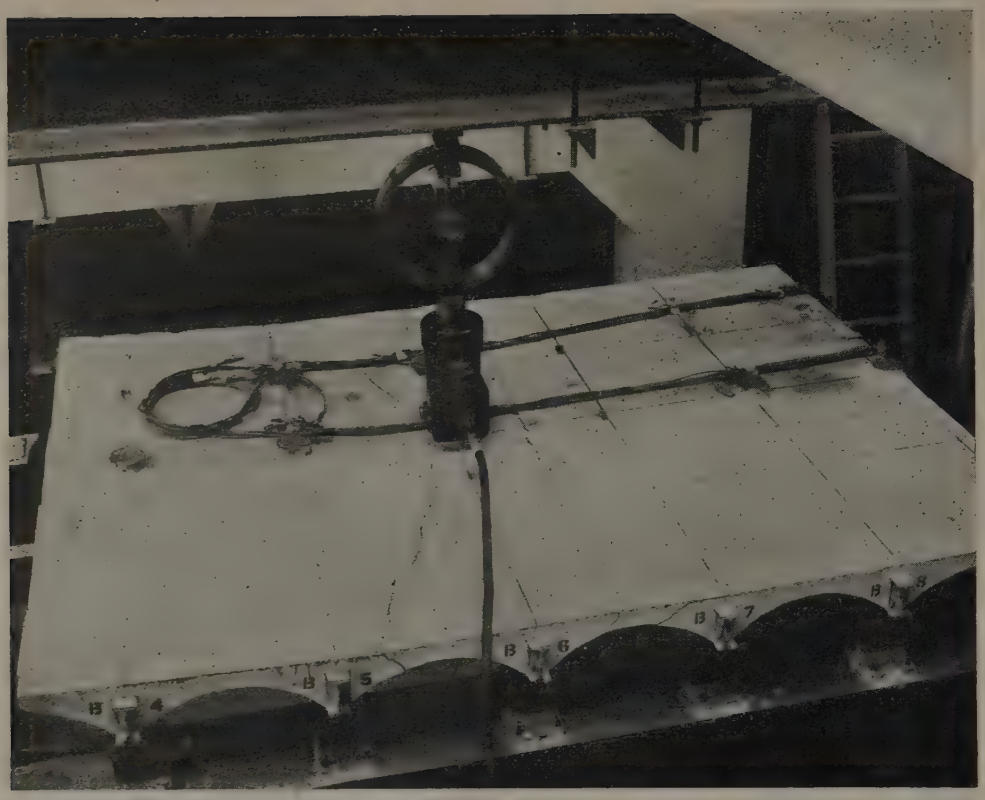
The Differential Analyser

THE work of the Building Research Station of the Department of Scientific and Industrial Research, and particularly that of the Engineering and Soil Mechanics Divisions, is well known to many members of the Institution. Important results of research work carried out by members of the Station's staff have, indeed, from time to time first been publicized in papers read before the Institution and in articles printed in THE STRUCTURAL ENGINEER. The Station carries on an extensive information service, both through its publications and by answers to specific enquiries addressed to the Director by public bodies and by individuals, including many Chartered Structural Engineers. Parties of members of various organizations, professional, trade and educational, are regularly conducted round the Station, by appointment. In a general tour during working hours, however, parties have to be limited in size, the time spent in any one laboratory is brief, and the visitor must usually take his chance as to whether any item of work which he particularly wishes to see in progress is at an interesting stage at the time of his visit.

The opportunity afforded to visitors by the Open Days arranged on the 18th and 19th July was unprecedented and highly appreciated. The occasion had been widely publicized in appropriate quarters, and a gratifying number of prospective visitors applied for tickets of admission. 434 attended on the first day, 505 on the second, and an overflow session had to be held on the 21st. Specimens had been prepared long in advance to be ready for test on the day (no small matter of organization in itself), and the staff engaged on each piece of work were present to demonstrate and to answer questions. Specimens already tested were on display, with photographs, graphs and descriptive paragraphs mounted on display screens, to explain the object of the investigation and the way in which it was being carried out.

Much of the early work of the Station was devoted to determining the characteristics of materials new and old, also to obtaining data for British Standards and Codes of Practice. This is still needed and is still carried on, but the emphasis has now moved rather towards the testing of composite structures or parts of structures, and the interaction of connected members. Examples of such combinations on which work is in progress are : encased steelwork, frames stiffened by panel filling, shell roofs, prestressed concrete members, slabs with stiff and with flexible beams at their edges, the distribution of the effect of a local load between the component members of a bridge deck system, the behaviour of structural combinations when the movement in places is beyond the range of elastic strain. In this work minute movements at many points in a specimen must be measured and recorded. Strain gauges of various makes are in use, including several devised and developed by the Station's own Instrument Section, such as the now familiar acoustic strain gauge, and the new " squat " gauge which can be inserted in a gap only 1½ in. wide between, say, a hydraulic jack



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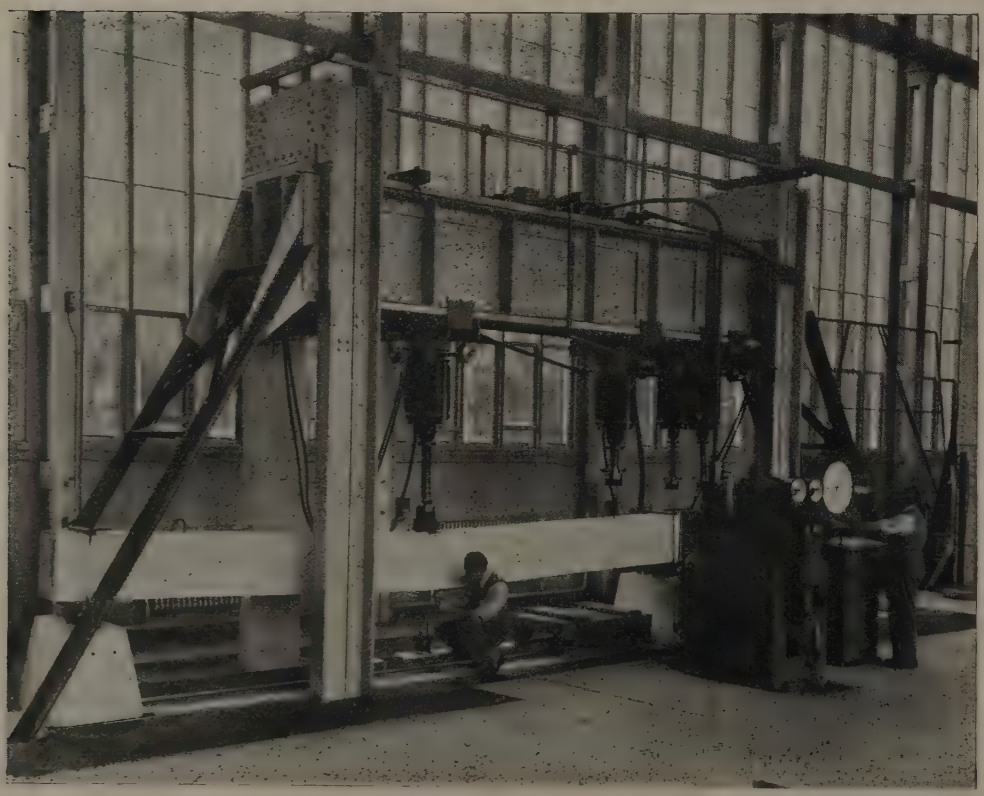
A test to failure of a model jack-arch bridge built to a scale of about 1:5. Its behaviour proved to be more complex but for design purposes it was shown to be accurate enough to consider the deck as an empirically determined equivalent slab-and-girder deck.



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Reinforced concrete beam under static loading test

d the body against which it is pressing. The pressure measured by means of an electrical resistance gauge ed to the wall of a tubular ring which is subjected to squashing movement by the load. The interpretation experimental work on combined structures such as ilding frames in terms of theory involves considerable ill in mathematics and much arduous computation. deal with the latter an ingenious Differential nalyser has been built in the Stations workshops. It as during the display set to integrate differential uations expressing the behaviour of a column which is bent until part of it is deforming plastically, a very dious problem to solve by analytical methods, but sily dealt with by the machine when presented in ometrical terms. Graphs to facilitate the design of eel building frames on a more accurate and economical asis than that used in current practice have already een issued. Tests were shown in progress on the rength of large concrete pipes, and methods of joint-g them, and experiments are being made to determine hat forms of laboratory test would best represent the cissitudes through which such pipes must pass during andling, laying, backfilling and service.

Experimental work on plastic yield and on fatigue, he latter now carried on in a hydraulically-operated ulsating machine recently installed, is aimed at clari-ing various problems in design on which current odes of Practice give less guidance than could be esired. Work on full-size columns and walls, now rried out in a 500 ton Amsler machine, will shortly e supplemented and extended when a similar machine f 1,000 tons capacity now on order is delivered.

Valuable work, much of it of a pioneering nature, was done in the Soil Mechanics Division before this branch of study had received recognition in Britain. This work was vital in contributing much of the information on which Civil Engineering Code of Practice No. 4, " Foundations," was based. Now many other laboratories, industrial and university, are carrying on the work, but the Station's Laboratory is still well to the fore. Practically the whole of its available resources were for a time concentrated on the problems of stability and seepage raised by the disasters which befell the sea defences of the East Coast of Britain in 1953. This work is still in progress. So is work on forecasting the probable, and subsequent measurement of the actual, settlement of the foundations of a number of important structures.

Open days on the scale of that recently held can only be occasional : work must go on, often against time and with a staff which, like that of nearly all organizations now employing scientists, is not nearly large enough for the work which needs to be done. Even at the cost of some interference with the work, however, to arrange from time to time a shop window, in which the industry and the public for whose ultimate benefit the work is undertaken can see what is being done, can only do good in helping to shorten the lag between the acquisition and the application of new knowledge. It is therefore to be hoped that these highly successful Open Days are only the first of many, and that from time to time the work of other Divisions will be shown in the same informative manner.

Institution Notices and Proceedings

IMPORTANT NOTICE TO SUBSCRIBERS

Subscribers, Newsagents and Subscription Agencies at home and abroad are asked to note that as from the 1st January, 1955, copies of THE STRUCTURAL ENGINEER can only be obtained on application to the Secretary of the Institution of Structural Engineers, 11, Upper Belgrave Street, London, S.W.1.

Subscription Rate

The Annual Subscription, post free, to any part of the world is 44s. 6d.

Single Copies

The price of single copies is 3s. 6d. or 3s. 9d. if sent by post.

In order to fulfil all demands, subscribers and others are requested to state their requirements as soon as possible.

The usual rate of discount will apply.

PRESIDENTIAL ADDRESS—SESSION 1955-56

A General Meeting of the Institution of Structural Engineers will be held at 11, Upper Belgrave Street, London, S.W.1., on Thursday, 6th October, 1955, at p.m., when Mr. Stanley Vaughan, B.Sc., M.I.C.E., M.I.Struct.E., A.C.G.I., will be installed as President for the Session 1955-1956, and will give the Presidential Address.

FORTHCOMING MEETINGS

Thursday, October 13th, 1955

Ordinary Meeting at 11, Upper Belgrave Street, London, S.W.1., at 6 p.m., when Professor A. J. Ockleston, B.E., Ph.D., D.Sc.(Eng.), M.I.C.E., M.I.Struct.E. (Vice-President) will give a paper on " Load Tests on a Three Storey Reinforced Concrete Building in Johannesburg."

Friday, 14th October, 1955

Joint Meeting with the Cement and Concrete Association, the Prestressed Concrete Development Group and the Reinforced Concrete Association, when Professor Pier Luigi Nervi will lecture on " Concrete and Structural Form."

This meeting was to have been held earlier in the year but was postponed owing to the illness of the Lecturer. In view of the large attendance expected, the meeting will be held at the Friends Meeting House, Euston Road, London, N.W.1. Arrangements have been made to reserve 200 seats for members of the Institution and to ensure that accommodation will be available, it is suggested that members should apply to the Secretary of the Institution for reserved tickets. The meeting will commence at 6 p.m., and will be followed at 8 p.m. by a Dinner at the Hungaria

Restaurant. Those wishing to attend the Dinner should apply for particulars to Mr. P. H. Gooding, the Cement and Concrete Association, 52, Grosvenor Gardens, S.W.1.

Thursday, October 27th, 1955

Ordinary General Meeting for the election of members at 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when a Discussion on Ultrasonic Testing of Concrete will be held. Dr. R. Jones, of the Road Research Laboratory, will open the discussion, which will be preceded by a demonstration of the apparatus and techniques.

Thursday, November 24th, 1955

Ordinary General Meeting for the election of members at 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when a Discussion on the Report of the Ad Hoc Committee on Structural Safety will be opened by the Chairman and members of the Ad Hoc Committee.

Thursday, December 15th, 1956

Ordinary General Meeting at 5.55 p.m., followed at 6 p.m. by a Joint Meeting with the British Section of the Societe des Ingenieurs Civils de France, when Monsieur R. Giguet will give a paper on " The Tignes Hydro-Electric Scheme."

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS—JANUARY, 1956

The Examinations of the Institution will next be held at centres in the United Kingdom and Overseas on January 10th and 11th, 1956 (Graduateship) and January 12th and 13th (Associate-Membership).

HONOURS AND AWARDS

In offering their sincere congratulations to the following members on the distinctions recently conferred upon them the Council feel they are also expressing the good wishes of the Institution :—

Order of the British Empire—O.B.E. (Military Division) and Efficiency Decoration, Lt.-Col. G. A. Lindell, D.S.O. (Associate-Member).

Honorary D.Eng., Sheffield University, Professor J. Husband (Past President).

RESEARCH AWARDS

The Council have instituted a Research Prize Fund, from which awards may be made annually to the author or joint authors of papers describing original research which they have carried out. Research awards may be made for papers read at Headquarters or in the Branches and published in the Journal, or for papers published in the Journal only without being read at an open meeting.

The assessment for such awards will be made annually, but awards will be made only to the contributors of such papers as reach a standard judged by the Literature Committee to be satisfactory.

Work submitted under this scheme must be original and may include any of the following :

(a) investigations of an experimental or analyti character ;
(b) studies of historical or statistical records ;
(c) improvements in principles or methods of c struction ;
(d) research into methods of structural engineer and building, the nature and use of plant and organisation of engineering work ;
(e) any related or combined studies which deemed by the Literature Committee to be o research character.

In cases where the research work described in paper was not the work of one individual, the names all the collaborators should be given in the paper.

Awards may take any or all of the following form A research medal ; a diploma ; a money prize.

Application for consideration for a research aw must be made to the Secretary of the Institution, a in preparing papers for reproduction in the Jour authors must comply with the conditions laid down all such contributions. Particulars of these conditi may be obtained from the Secretary.

In judging research papers, the following factors be considered :

(a) the nature of the subject and its conclusions ;
(b) the value of the paper in advancing the scie and art of structural engineering ;
(c) the standard of preparation and orderly arran ment of the subject-matter.

Research papers will also be eligible for adjudicat for the Institution Medals if they comply with Regulations governing those awards.

The closing date for the receipt of applications respect of papers published in the Journal betw October, 1954, and September, 1955, is October 1955.

LONDON GRADUATES' AND STUDENTS' SECTION

The first indoor meeting of the Section will be h at 11, Upper Belgrave Street, on Monday, 10th Octo at 6.30 p.m., when Mr. K. Severn, M.C., M A.M.I.C.E. (Associate-Member of Council) will add the Section on a subject of general interest.

This will be followed by a smoking concert w the Committee hopes to be able to meet as m members as possible, particularly new member

Hon. Secretary : D. E. Capelin, 40, Thorn Crescent, Old Coulsdon, Surrey.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Thursday, October 20th, 1955

The inaugural meeting of the Session will be h at the Reynolds Hall, College of Technology, M chester, at 7.15 p.m., when the Chairman's Add will be given by Mr. J. H. Morris, M.I.Struct.E. the conclusion of the Address, films of general inte will be shown. The President and the Secretary the Institution will attend the meeting.

Wednesday, November 9th, 1955

Joint Meeting with the Liverpool Metallurg Society, at Picton Hall. Dr. A. A. Wells on " S and Welding."

Monday, November 14th, 1955

At the College of Technology, Manchester. Mr. Bernard Godfrey, A.M.I.C.E., A.M.I.Struct.E., " Joints in Steel Rigid Frameworks."

Wednesday, November 30th, 1955

At the Liverpool Engineering Society, Dale Street, verpool. Mr. J. Singleton-Green, M.Sc., M.I.C.E., M.I.Mech.E. (Member of Council) on " Concrete an Engineering Material."

Thursday, December 8th, 1955

At the College of Technology, Manchester. Mr. M. Little, B.Sc.(Tech.), A.M.I.C.E., A.M.I.Struct.E. " The Influence of the Motor Vehicle on Urban ildings."

Unless otherwise stated, the above meetings will mmence at 6.30 p.m., preceded by tea at 5.45 p.m.

Joint Hon. Secretaries: J. L. Robinson, M.I.Struct.E., 314, Northenden Road, Sale, Manester; M. D. Woods, 58, Spring Gardens, Salford, 6.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Saturday, October 8th, 1955

Annual Dinner and Ladies Evening, at the Botanical ardens, Birmingham. The President and the Secreıry of the Institution will be present.

Friday, October 28th, 1955

At the James Watt Memorial Institute, Birmingham. hairman's Address, by Mr. W. D. Christie, M.I.Struct.E. isit of the President and the Secretary of the Initution.

Tuesday, November 1st, 1955

Joint Meeting with the Reinforced Concrete Associion, Midland Counties Branch, at the Midland ıstitute, Birmingham. Mr. E. Shepley, B.Sc., .M.I.C.E., on " Design of a Building."

Thursday, November 3rd, 1955

At the Becket Sale Rooms, Derby, 7 p.m. Mr. W. Clenshaw, D.I.C., B.Sc.(Eng.), A.M.I.C.E., on Derby Station—Reconstruction of Platform Roofs."

Friday, November 25th, 1955

Joint Meeting with the Reinforced Concrete Assocition, Midland Counties Branch. Mr. J. E. C. Farerother, M.I.Struct.E., on " Prestressed and other Reinforced Concrete Work at Ansells Brewery."

Saturday, November 26th, 1955

Works visit to Ansells Brewery to inspect plain and restressed reinforced concrete work described in he paper by Mr. Farebrother. Assemble at 10.30 .m.

Unless otherwise stated, meetings will commence t 6 p.m.

Hon. Secretary: L. A. Firminger, A.M.I.Struct.E., 56, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES GRADUATES' AND STUDENTS' SECTION

Hon. Secretary: J. E. Jeffries, 18, Radnor Road, Handsworth, Birmingham, 23.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, October 4th, 1955

At Middlesbrough. Mr. E. A. Parsons, M.I.Struct.E. (Chairman-Elect) on " Relationship between Structural and Civil Engineering."

Wednesday, October 12th, 1955

At Newcastle. Installation of the Chairman, Mr. E. A. Parsons, M.I.Struct.E., who will give an address on " Relationship between Structural and Civil Engineering." Visit of the President and the Secretary of the Institution. This meeting will be held in the Northern Gas Board Demonstration Theatre, Grainger Street, Newcastle-on-Tyne.

Tuesday, November 1st, 1955

At Middlesbrough. Mr. H. Saunders, A.I.Struct.E., on " Castellated Construction."

Wednesday, November 2nd, 1955

The above meeting will be repeated at Newcastle.

Unless otherwise stated, Middlesbrough meetings will be held at the Cleveland Scientific and Technical Institution, Corporation Street, Middlesbrough, and Newcastle meetings at the Neville Hall, Westgate Road, Newcastle-upon-Tyne. Meetings will commence at 6.30 p.m., preceded by buffet tea at 6 p.m.

Hon. Secretary: O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, October 4th, 1955

Opening Meeting. " The Chairman presents . . . " Meeting to be arranged by the Chairman, Mr. Robert Ferguson, B.A., B.E., M.I.C.E., M.I.Struct.E.

Tuesday, November 1st, 1955

Lecture to be arranged.

Tuesday, December 6th, 1955

Mr. C. Munro, A.R.I.B.A., on " The Architect and the Structural Engineer."

Meetings will be held at the College of Technology, Belfast, at 6.45 p.m., and will be preceded by tea at the Overseas League premises, Wellington Place, Belfast, at 6 p.m.

Hon. Secretary: A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., " Barbizon," 2, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Monday, October 10th, 1955

At Ca'doro Restaurant, Glasgow, at 6 p.m. Chairman's Address by Professor W. T. Marshall, B.Sc.(Eng.) Ph.D., M.I.C.E. M.I.Struct.E., A.C.G.I., D.I.C. Visit of the President and the Secretary of the Institution.

Tuesday, October 11th, 1955

The Annual Dinner and Dance will be held at Burlington House, Bath Street, Glasgow. The President and the Secretary of the Institution will be present.

Tuesday, November 8th, 1955

Joint Meeting with the East of Scotland Branch of the Institute of Welding, at the Adelphi Hotel, Coburn Street, Edinburgh, at 6.15 p.m. Mr. James McLean, B.Sc., on " Welded Plate Structures."

Tuesday, December 6th, 1955

At the Institution of Engineers and Shipbuilders, Elmbank Crescent, Glasgow, 7 p.m. Mr. Donovan H. Lee, B.Sc.(Eng.), M.I.C.E., M.I.Mech.E., M.I.Struct.E., M.Am.Soc.C.E., on " The Advantages and Disadvantages of Structural Steelwork, Reinforced Concrete and Prestressed Concrete."

Hon. Secretary: W. Basil Scott, M.I.Struct.E., 19, Waterloo Street, Glasgow, C.2.

SOUTH WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, November 5th, 1955

At the Duke of Cornwall Hotel, Plymouth, 6.30 p.m. Chairman's Address by Lt.-Colonel R. Hazzledine, O.B.E., T.D., M.I.Struct.E. The President and the Secretary of the Institution will attend the meeting, which will be followed by an Informal Dinner.

Friday, December 16th, 1955

Meeting at Exeter, when a paper on Timber Construction will be given.

Joint Hon. Secretaries: E. W. Howells, M.I.Struct.E., 10/12, Market Street, Torquay, Devon. C. J. Woodrow, J.P., " Elston," Hartley Park Villas, Tavistock Road, Plymouth.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Tuesday, October 25th, 1955

Chairman's Address, at the Mackworth Hotel, Swansea, 6.30 p.m. Mr. A. V. Hooker, M.I.Struct.E., A.M.I.C.E., will give the Chairman's Address : " Recent Trends in the Design of Industrial Buildings."

Thursday, November 1st, 1955

At the South Wales Institute of Engineers, Park Place, Cardiff, 6.30 p.m. The Chairman's Address will be repeated.

Saturday, November 12th, 1955

At the County Buildings, Colwyn Bay, 6 p.m. The Chairman's Address will be repeated.

Thursday, December 15th, 1955

At the South Wales Institute of Engineers, Cardiff, 6.30 p.m. Joint Meeting with the Institution of Civil Engineers. Mr. F. Walby, M.I.C.E. and Mr. H. C. Adams, M.I.C.E., on " Prestressed Concrete as applied to Building Frames."

Hon. Secretary: K. J. Stewart, A.M.I.C.E., A.M.I.Struct.E., 15, Glanmor Road, Swansea, Glam.

WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, October 7th, 1955

Mr. N. G. T. Ball, M.I.Struct.E. will give the Cha man's Address. Visit of the President and the Sec tary of the Institution.

Thursday, November 17th, 1955

Combined Meeting with the Institution of Ci Engineers. Mr. C. Parry on " Shuttering."

Friday, December 2nd, 1955

Papers by Branch members :—Mr. J. A. Youn A.M.I.C.E., A.M.I.Struct.E., on " Joints in Wat Retaining Structures with particular reference reservoirs," and Mr. N. T. A. Beavan, A.M.I.Struct.I on " Co-operation between the Consultant and t Steelwork Engineer."

Friday, December 9th, 1955

Combined Dance, Royal Hotel, Bristol.

Unless otherwise stated, meetings will be held in t Small Lecture Theatre in the new Engineering Scho University of Bristol, at 6 p.m., preceded by tea 5.30 p.m.

Hon. Secretary: E. Hughes, M.I.Struct.E., Southdown Road, Westbury-on-Trym, Bristol,

YORKSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, October 19th, 1955

Professor R. H. Evans, D.Sc., D. es Sc., Ph. M.I.C.E., M.I.Mech.E., M.I.C.E.I., M.I.Struct.E., w give the Chairman's Address. The President and e Secretary of the Institution will attend the meeti

Wednesday, November 16th, 1955

Joint Meeting with the Yorkshire Association of the Institution of Civil Engineers, at the Universi Leeds, 6.30 p.m. Mr. J. A. Banks, O.B.E., M.I.C. on " The Allt-na-Lairige Prestressed Concrete Dar

Wednesday, December 21st, 1955

Mr. J. Robertshaw, B.Sc., A.M.I.C.E., on " Geop sical Methods of Exploration and their applicat n to Structural Engineering Problems."

Unless otherwise stated, meetings will be held t the Great Northern Hotel, Leeds, at 6.30 p.m.

Hon. Secretary: E. Wrigley, A.M.I.Struct.E., 7 The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary: A. E. Tait, B.Sc., A.M.I.C A.M.I.Struct.E., P.O. Box No. 3306, Johannesbu South Africa.

During weekdays Mr. Tait can be contacted in t City Engineer's Department, Town Hall, Johannesbu Phone 34-1111, Ext. 257.

Natal Hon. Secretary: E. G. Bennett, A.M.I.Struct c/o The Reinforcing Steel Co. Ltd., P.O. Box 9 Merebank, Durban.

Cape Section Hon. Secretary: R. Stubbs, M.I.Struct African Guarantee Building, 8 St. George's Str t Cape Town.

Ultimate Strength of Axially Loaded Columns Reinforced with Square Twisted Steel and Mild Steel

By Professor R. H. Evans, D.Sc., M.I.C.E., M.I.Struct.E. and K. T. Lawson, B.Sc.

Introduction

In THE STRUCTURAL ENGINEER (July, 1948) Hajnal-Kónyi described tests on concrete columns reinforced with square twisted steel (T.S.) and mild steel (M.S.) The tests showed that columns with T.S. have substantially higher failure loads than those with M.S. and may undergo large deformations before collapsing. Adequate warning of failure is given by gradual cracking of the concrete whereas normal tied columns reinforced with M.S. often exhibit a brittle failure, which is undesirable in any structure.

The ultimate load of a column reinforced with M.S. is given by the following expression :

$$W = A_c \cdot C_p + A_s \cdot Y \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where W = ultimate load
A_c = area of column less area of steel
C_p = strength of plain concrete (often quoted as $\frac{2}{3}$ cube strength)
A_s = area of steel
Y = yield stress of M.S.

The ultimate loads of the columns reinforced with T.S. fitted the expression :

$$W = A_c \cdot C_p + A_s \cdot \beta\rho \quad . \quad . \quad . \quad . \quad . \quad (2)$$

where $0.75 \leq \beta \leq 1.0$

and ρ = proof stress of the steel (defined as the stress corresponding to a strain of 0.005)
= 70,000 psi in these tests.

The behaviour of columns reinforced with T.S. has already been described by Hajnal-Kónyi but it will be helpful to describe the general theory again here.

Fig. 1 gives stress-strain curves obtained from tests on short concrete cylinders (2). It can be seen that maximum concrete strength is developed at strains which may vary from 0.0013 for low and medium grade concretes, to about 0.002 for high strength concretes. This strength can be maintained for strains up to 0.0025 but then decreases, until failure occurs at strains greater than 0.003. For the same range of strains the stress in T.S. increases continuously, as shown by Fig. 2.

A study of Figs. 1 and 2 shows that the load on a reinforced column may be expected to increase until a strain of 0.0025 is reached. Whether or not the load will increase at higher strains depends upon the relative strength of the concrete and reinforcement i.e. on the value $\alpha = \frac{A_s \cdot \rho}{A_c \cdot C_p}$. The ultimate load is reached when the rate of decrease of concrete load equals the rate

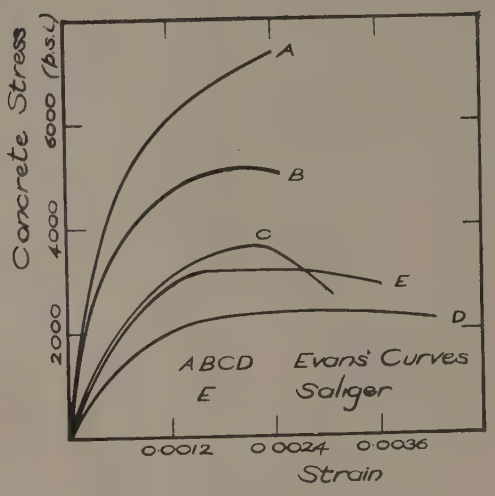

Fig. 1

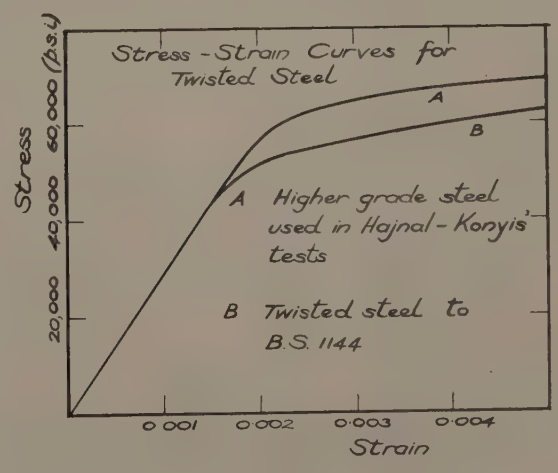

Fig. 2

of increase of steel load. Thus for a lightly reinforced column of high strength concrete (for which α is small) the load-strain curve for the steel is relatively flat and no increase in load may be expected for strains greater than 0.0025. For a column with a heavy reinforcement in low strength concrete (for which α is large) the concrete curve is relatively flat, so that the decrease of concrete load is small. In this case it may be expected that the load-bearing capacity of the column will increase considerably for strains well beyond 0.0025.

The steel stress developed in a column may be expressed as a fraction β of the proof stress, such that β depends upon the value α. The following relationships in Table 1 were found by Hajnal-Kónyi from tests on columns.

Table 1

α	β
0.65	0.75
1.4	0.9
3.0	1.0

However, Hajnal-Kónyi's paper while dealing fully with the results obtained was limited in its scope. The total number of columns tested was 22 and out of these eight were plain, four were reinforced with M.S. and ten were reinforced with T.S. Two concrete strengths of 2,250 psi. and 5,000 psi. cube strength and two percentages of steel, namely 3.52 per cent. (four $\frac{3}{4}$ ins. T.S.) and 7.05 per cent. (eight $\frac{3}{4}$ ins. T.S.) were the chief variables in the tests. The Code of Practice (C.P. 114) published just after Hajnal-Kónyi's paper permits percentages of reinforcement from 0.8 per cent. to 8 per cent. The quality of the reinforcement used in the tests was somewhat better than the minimum required by the Code, the reinforcement being a medium tensile steel work-hardened by twisting to give a proof stress of 70,000 psi. The Code requires a proof stress of only 60,000 psi. Furthermore two percentages of links were provided in the columns, those with eight bars having links at 3 ins. pitch and those with four bars having links at 6 ins. pitch. This is unfortunate as it is now known that links at close spacings can increase the ultimate strains of a column. These higher strains are beneficial for T.S. since an increase of strain enables higher steel stresses to be developed in the columns before failure.

Table 2. Test Programme

Group	Column No.	Target Cube Strength (p.s.i.)	Steel	% Steel	Link Spacing	Column Size
A	1,1(a)	5000	4-$\frac{5}{8}$" S.T.	1.08	8"	12" x 12"
	2,2(a)	5000	4-$\frac{5}{8}$" S.T.	2.44	8"	8" x 8"
	3,3(a)	5000	4-$\frac{7}{8}$" S.T.	4.78	8"	8" x 8"
	4,4(a)	5000	4-1$\frac{1}{8}$" S.T.	7.92	8"	8" x 8"
	5,5(a)	3000	4-$\frac{5}{8}$" S.T.	1.08	8"	12" x 12"
	6,6(a)	3000	4-$\frac{5}{8}$" S.T.	2.44	8"	8" x 8"
	7,7(a)	3000	4-$\frac{7}{8}$" S.T.	4.78	8"	8" x 8"
	8,8(a)	3000	4-1$\frac{1}{8}$" S.T.	7.92	8"	8" x 8"
B	1,1(a)	5000	4-$\frac{5}{8}$" M.S.	1.08	8"	12" x 12"
	2,2(a)	5000	4-$\frac{5}{8}$" M.S.	2.44	8"	8" x 8"
	3,3(a)	5000	4-$\frac{7}{8}$" M.S.	4.78	8"	8" x 8"
C	1	5000	4-$\frac{5}{8}$" S.T.	1.08	8"	12" x 12"
	2	5000	4-$\frac{5}{8}$" S.T.	2.44	8"	8" x 8"
	3	5000	4-$\frac{7}{8}$" S.T.	4.78	8"	8" x 8"
	4	5000	4-1$\frac{1}{8}$" S.T.	7.92	8"	8" x 8"
D	9,9(a)	3000	4-1$\frac{1}{8}$" S.T.	7.92	3"	8" x 8"
	10(10a)	5000	8-$\frac{5}{8}$" S.T.	4.88	8"	8" x 8"
E	2,2(a)	5000	4-$\frac{5}{8}$" S.T.	2.44	8"	8" x 8"
	3	5000	4-$\frac{7}{8}$" S.T.	4.78	8"	8" x 8"

Notes.—Groups A, D and E were made by the Square Grip Reinforcement Co. Ltd.

Groups B and C were made by Messrs. Greenwoods of Leeds.

For these reasons it was necessary to carry ou further tests and the results obtained are presented i this paper.

Test Programme

Details of the 64 test columns are contained i Table 2. The capacity of the 200 ton testing machin in the Civil Engineering Laboratories of Leeds Univer sity limited the size of the columns to 8 ins. squar initially, but later a larger hydraulic jack was installed capable of giving loads up to 350 tons, and this enable some columns 12 ins. square to be included. Th overall length of the columns was 48 ins.

To establish the relationship between concrete prism strength and cube strength it was decided to cast plain column and twelve 6 ins. cubes with each re inforced column. Two concrete strengths were chosen namely 3,000 psi. and 5,000 psi. cube strength.

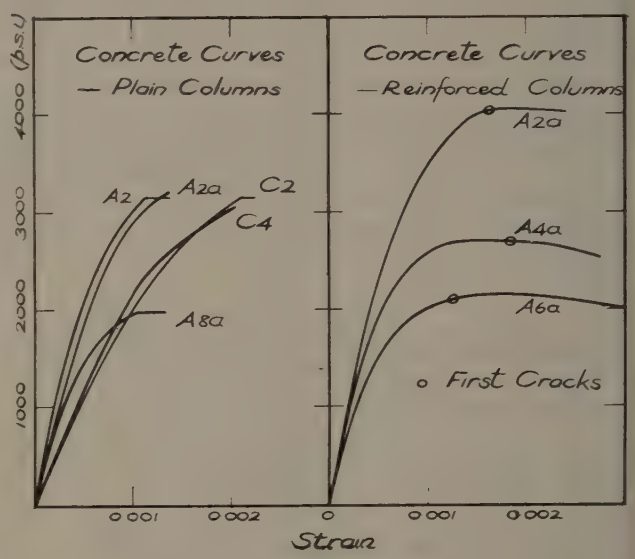

Fig. 3

The reinforcement in each column of group A con sisted of four bars of $\frac{5}{8}$ in., $\frac{7}{8}$ in. or 1$\frac{1}{8}$ ins. T.S. steel Binder pitch was purposely kept wide for reasons given previously. The Code requires that binder pitch sha not be greater than the least of the following :

(a) 12 ins.

(b) the least lateral dimension of the column

(c) 12 x diameter of the smallest vertical bar

Thus the maximum permissible binder pitch for th test columns came to 8 ins. The columns of group A were made by the Square Grip Reinforcement Co (Sunbury) Ltd. The concrete mixes were 1 : 2.4 : 3. and 1 : 1.92 : 2.88 for cube strengths of 3,000 psi. an 5,000 psi. respectively. The water/cement ratios wer 0.6 and 0.5 respectively.

For comparison purposes a further series of columns group B, were made with plain square M.S. reinforce ment in one concrete strength of 5,000 psi. Th columns of this series were made by Messrs. Georg Greenwood and Sons, Leeds, and the aggregate an mix proportions were different from series A. A stif mix of proportions 1 : 1$\frac{1}{2}$: 3 was used. This gave concrete with very different stress-strain propertie from those of the first series. Fig. 3 shows stress-strai

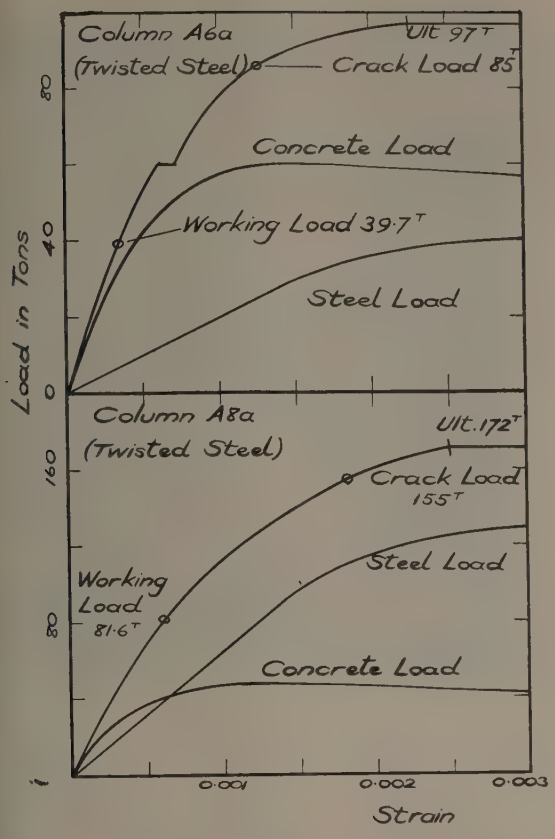

Fig. 4

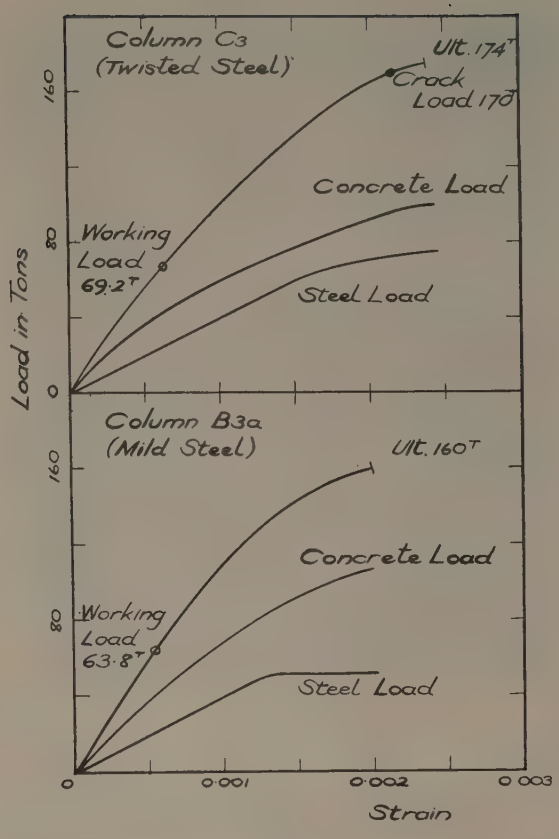

Fig. 5

diagrams obtained from plain columns of each concrete type. Concrete of group B had an unusually low modulus and did not develop its full strength before a strain of 0.0022 was reached. Concrete of group A was typical of most concretes of this strength; ultimate strength was developed at a strain of 0.0013.

Some further tests were carried out to demonstrate the behaviour of T.S. in the softer type of concrete. These tests formed group C and were also made by Messrs. George Greenwood and Sons.

Groups D and E consist of a few miscellaneous columns. Columns 9 and 9a had a binder pitch of 3 ins.; columns 10 and 10a had eight vertical bars instead of the usual four. The reinforcement for group E was T.S. of similar quality to that used in the tests made for Hajnal-Kónyi. Originally it was planned to do a complete series with this type of steel but the comprehensive results obtained so far made this unnecessary.

Testing

End Conditions

The reinforcement was cut equal to the full length of the columns so that the ends of the bars formed a level surface with the concrete. To ensure even bearing against the loading plates of the machine it is essential to have the ends of the columns quite flat, and therefore a great deal of time was spent grinding the ends of the columns to achieve this condition.

Some previous investigators[3 4] have used soft packings of strawboard or 'plaster of paris' to bed the columns into the machine. In order to avoid the time and labour spent on grinding this was tried in some of the earlier tests, but without success. The columns tested with packings failed at lower loads than those tested in the normal way. The following observations were made for columns tested with packings:

Table 3

Column	Cube Strength (p.s.i.)	Steel	Ult. Load W (tons)	Strain at Ult. Load	Calculated Load Wt	$\frac{W}{Wt}$
3	4920	4.$\frac{7}{8}$"T.S.	155	0.0021	168.8	0.92
3a	5100	4.$\frac{7}{8}$"T.S.	135	0.0034	173.1	0.78
7	2795	4.$\frac{7}{8}$"T.S.	119	0.0032	123.5	0.96
7a	3320	4.$\frac{7}{8}$"T.S.	120	0.0034	133.6	0.90
					Average	0.89

Columns tested with packings failed at loads which average 10 per cent. below the theoretical loads. A somewhat surprising feature of the tests was the high strains observed before the ultimate loads were reached. It is shown later that similar columns tested without packings reach ultimate load when the strains are about $\frac{2}{3}$ those given in Table 3. Examining the columns and packings after testing revealed that the steel was 'proud' of the concrete and had made indentations in the packings. Obviously the steel had failed in bond at the ends of the columns. The steel is therefore subject to a lower strain than the concrete and hence the expected stresses were not developed. No packings were used for the remainder of the tests and those tests in which packings were used were repeated. The results of previous investigations in which packings were used should therefore be regarded with suspicion.

Method of Testing

The strain at mid-height of each of the four sides of a column was measured by dial extensometers of 8 ins. gauge length. A more accurate measurement of strain was made by means of an extensometer of 30 ins. gauge length which recorded the mean column strain with a magnification such that a reading of 1 division on the dial was equivalent to a strain of 1.66×10^{-6}.

Load was transmitted through a dynamometer which registered the load to within 1.0 per cent. Care was taken in placing a column centrally in the machine but even so the applied load was sometimes found to be slightly eccentric. When this happened it was necessary to unload and adjust the position of the column or dynamometer until the readings of the extensometers agreed with one another, showing that the load was centric. Load was increased in increments of 5, 10 or 20 tons, depending on the strength of the column being tested. As failure load was approached the loading increments were reduced to $2\frac{1}{2}$ or 5 tons. Recording the extensometer readings at each stage of load usually occupied about two or three minutes.

Test Results

The chief test results are contained in Tables 4 and 5 and some typical load-strain curves are given in Figs. 4 and 5.

Table 4. Results for Reinforced Columns

Group	Column	Cube Strength Cu (p.s.i.)	Proof Stress (p.s.i.)	Load at 1st Crack P (tons)	Strain at 1st Crack	Ultimate Load W (tons)	Strain at Ultimate Load	$\frac{P}{W}$	Maximum Observed Strain
A	1	6700	65,000	300	0.0016	350	(2)	0.86	0.0023
	1a	6800	66,000	300	0.001	350	0.0023	0.86	0.0023
	2	4930	67,000	120	0.0015	130	0.002	0.92	0.0023
	2a	5500	68,000	140	0.0015	150	0.002	0.93	0.0024
	3	5900	64,000	175	0.0016	185	0.002	0.95	0.002
	3a	4100	69,800	125	0.0014	142	0.0023	0.88	0.0023
	4	4670	66,000	186	0.0017	200(1)	0.0024	0.93	0.0024
	4a	3900	68,000	184	0.0018	196	0.0023	0.94	0.0028
	5	3660	67,000	165	0.0013	187	(2)	0.88	
	5a	4400	67,000	215	0.0014	230	0.0025	0.935	0.0027
	6	2790	69,000	90	0.0014	95	0.0022	0.95	0.0028
	6a	2940	61,000	85	0.0013	97	0.002	0.87	0.003
	7	3400	64,000	110	0.0015	131	0.0023	0.84	0.003
	7a	3260	68,000	110	0.0015	136	0.0023	0.81	0.0023
	8	3130	65,000	150	0.0016	178	0.0025	0.84	0.003
	8a	3100	64,000	155	0.0018	172	0.0025	0.90	0.003
B	1	5950	40,000	280	0.0021	280	0.0021	1.0	0.0021
	1a	5300	40,000	260	0.002	260	0.002	1.0	0.002
	2	5300	40,000	130	0.0023	130	0.0023	1.0	0.0023
	2a	5600	40,000	140	0.0021	140	0.0021	1.0	0.0021
	3	4170	38,000	135	0.0022	135	0.0022	1.0	0.0022
	3a	5300	38,000	160	0.002	160	0.002	1.0	0.002
C	1	6700	62,000	310	0.0022	325	0.0025	0.95	0.0025
	2	4700	61,000	123	0.0022	125	0.0023	0.99	0.0023
	3	4700	61,000	170	0.0021	174	0.0024	0.98	0.0024
	4	4540	61,000	205	0.0027	212.5	0.0031	0.96	0.0031
D	9	2690	61,000	160	0.0022	180	(3)	0.89	0.0035
	9a	2700	61,000	135	0.0013	168	(3)	0.08	0.0035
	10	4300	62,000	130	0.0014	154	0.0024	0.85	0.0025
	10a	5950	66,000	185	0.0015	200	0.002	0.93	0.002
E	2	5290	71,000	120	0.0014	150	0.002	0.80	0.002
	2a	5400	71,000	130	0.0013	154	0.002	0.84	0.002
	3	4930	68,000	150	0.0013	170	0.002	0.88	0.0026

Notes.—(1) Ultimate load not reached
(2) Strain readings discontinued
(3) Strains exceeded range of instruments.

Table 5. Results for Plain Columns

Group	Column	Cube Strength Cu (p.s.i.)	Ultimate Load (tons)	Column Strength Cp (p.s.i.)	$\frac{Cp}{Cu}$	Maximum Observed Strain
A	1	6700	280	4360	0.65	0.0006 @ 180 tons
	1a	6800	270	4200	0.62	0.0006 @ 185
	2	4910	95	3320	0.67	0.0012 @ 90
	2a	5350	90	3150	0.59	0.0012 @ 90
	3	5900	120	4200	0.71	
	3a	None Tested				
	4	4600	98	3440	0.75	0.0008 @ 90
	4a	3900	88	3080	0.79	0.001 @ 80
	5	3660	155	2410	0.66	
	5a	3600	142	2210	0.62	
	6	2800	70	2450	0.87	0.0013 @ 70
	6a	2990	78	2760	0.92	0.0012 @ 78
	7	3400	70	2450	0.71	0.0007 @ 60
	7a	None Tested				
	8	3130	50	1750	0.56	0.0013 @ 50
	8a	2675	55	1940	0.67	0.0013 @ 55
B	1	5950	240	3700	0.62	
	1a	5300	230	3600	0.68	
	2	5300	90	3150	0.60	0.0015 @ 70
	2a	5600	112.5	3950	0.71	0.002 @ 102.5
	3	4170	75	2620	0.63	0.002 @ 75
	3a	5300	105	3680	0.70	0.0016 @ 100
C	1	None Tested				
	2	4700	87	3050	0.65	0.002 @ 87
	3	5530	125	4400	0.79	0.0021 @ 125
	4	4700	90	3150	0.67	0.0022 @ 90
D	9	3050	50	1750	0.57	0.0006 @ 35
	9a	2620	46	1610	0.62	0.001 @ 40
	10	4600	90	3150	0.68	
	10a	5950	126	4400	0.74	
E	2	5300	110	3860	0.73	0.0011 @ 90
	2a	5400	105	3680	0.68	0.0011 @ 90
	3	4960	105	3680	0.74	0.001 @ 90

Table 4 shows that the maximum strains observed were about 0.003. However it should be made clear that these are not ultimate strains; they were in fact measured while the column was still holding its ultimate load and were the last possible observations made before the column shortens very rapidly just before it collapses. A decrease in load of about 10 per cent of the ultimate was usually observed before the column failed. Many columns held their ultimate loads for several minutes, during which time a considerable shortening could be observed. If the ultimate load was reached when the load was being increased it was only possible to record the strains just before or at ultimate load.

Figs. 6 and 7 show the appearance of some columns after failure. If load is reduced immediately after the ultimate load is exceeded the column is badly cracked but still intact. Further loading eventually causes the concrete to shear and crush out between two binders where cracking has occurred. Local shortening then causes the main bars to buckle. Fig. 7 shows a M.S. column which has sheared across a plane inclined at about 70° to the horizontal. Some other columns also failed in this way.

Plain columns failed either by shearing diagonally or by shearing into a wedge which splits the surrounding concrete. Failure was liable to be violent, especially for higher strength concrete, and so strain measurements were discontinued in most tests before the ultimate load was reached.

Fig. 6

Fig. 7

The provision of a high percentage of links in columns and 9a, group D enabled these columns to undergo very large deformations. Strains of 0.0035 were measured at 97 per cent. of the ultimate load and which increased to about 0.005 at failure. Column 9a failed at the top against the bearing plate of the machine and the results for this column have been ignored in the analysis. The remaining columns of groups D and E behaved like columns of group A.

Analysis

The method of testing ensured that both concrete and steel were subjected to the same strain. The load strain curves for the reinforcement have been plotted in Figs. 4 and 5. It has been assumed that the steel stresses are initially zero but there will be some compressive stress caused by shrinkage of the columns whilst drying out. Shrinkage has no effect on the ultimate load of a column, but the maximum strains recorded are the net strains measured from the start of the test and not the gross strains. The load carried by the concrete is equal to the difference between the total load supported by the column and the load carried by the steel. Thus the load-strain curve for the concrete (marked Concrete Load) has been determined by plotting the difference between Column Load and Steel Load. In this way it has been possible to obtain load-strain relations for the concrete for strains up to 0.0033.

Fig. 3 shows that concrete stress develops in the reinforced columns in exactly the same way as in plain columns. Maximum strength is developed at strains of 0.0013 for group A, 0.002 for group B (see Figs. 4 and 5). The extended concrete lines compare to those obtained by Evans and Saliger. It is evident that concretes of medium strength may undergo considerable compression beyond the point at which maximum load is reached, and that there is only a slight reduction in stress for strains up to 0.003.

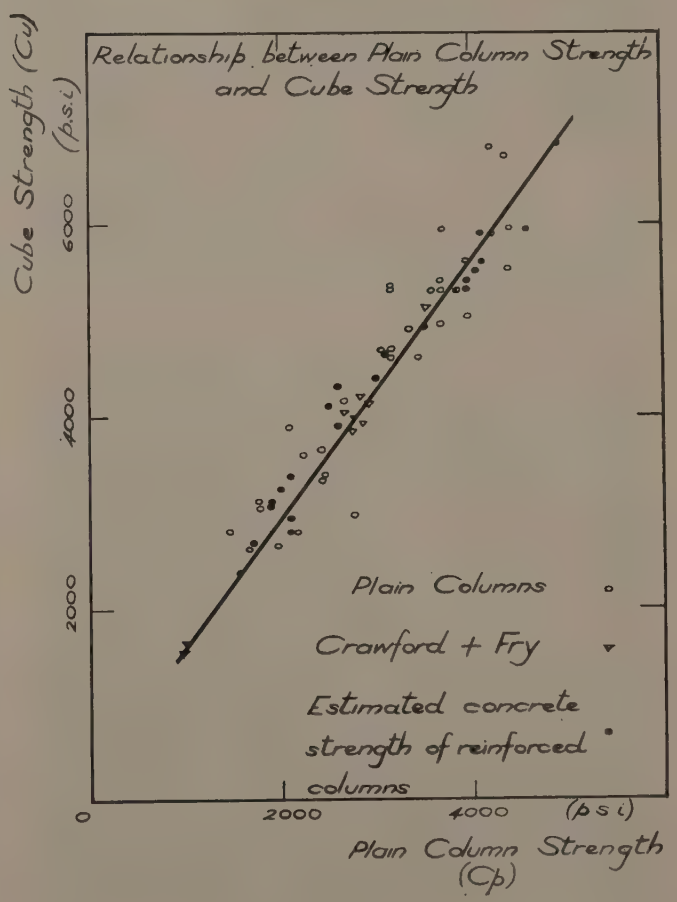

Fig. 8

Maximum concrete stresses derived for the reinforced columns have been plotted against the cube strengths in Fig. 8. To these have been added the results obtained for plain columns, and averaged results obtained from similar tests made by Crawford and Fry[5]. The results are all similar. The ratio $\frac{C_p}{C_u}$

appears to increase slightly for higher strength concrete, the average value for 3,000 psi. cube-strength being 0.67 and for 5,000 psi. cube-strength 0.70. An appreciable amount of scatter is present in the results; values from 0.6 to 0.8 are common. The Stuttgart tests[6] show similar variations for cube strengths up to 4,500 psi., but thereafter the average ratio decreases considerably. Talbot[7] found the strength of plain columns to be 70 per cent. of the strength of 12 in. cubes; King[3] has reported a similar value for 4 in. cubes. Cube size in the present tests was 6 ins., in the Stuttgart tests cube size was 8 ins. The size of plain columns in the tests mentioned varied from 8 ins. to 15 ins. square.

It has been assumed that the concrete strength of the reinforced columns is given by the mean curve of Fig. 8.

The stresses in the reinforcement must be estimated from the strains at which the reinforced columns reach their ultimate loads. Table 4 shows that columns of group A developed their ultimate loads at strains of 0.002 to 0.0025. According to Fig. 2 the steel stresses must have been about 54,000 psi.

Strains at ultimate load were slightly higher for columns of group C, especially for column 4, which developed ultimate strength at a strain of 0.0031. For columns 1-3 a steel stress of 55,000 psi. has been assumed and for column 4 a stress of 57,000 psi. Similarly a stress of 58,000 psi. is assumed for columns of group E, which have the higher grade of steel. For the M.S. columns the yield stress of the steel was developed and this must be used in the usual formula for such columns.

Column 9 group D developed strains greater than 0.0035 before the ultimate load was reached. The shape of the load-strain curve for this column indicates that a strain of 0.005 was developed at ultimate load. The steel stress at failure has been assumed to be 61,000 psi., the proof stress.

The formula for columns reinforced with T.S. is

$$W = A_c \cdot C_p + A_s \cdot C_s \quad . \quad . \quad . \quad . \quad . \quad (3)$$

The steel stresses (C_s) have been assumed as shown above. The theoretical loads have been calculated in Table 6. Fig. 9 shows how the calculated loads agree with the observed loads if a steel stress of 54,000 psi. is assumed.

Discussion

The T.S. columns cast in concrete of normal stress-strain characteristics as in group A, cracked well below the ultimate load. This can be seen clearly from the ratios $\frac{P}{W}$ in Table 4 which vary between 0.81 and 0.95. It should be borne in mind that the load was increased in increments of 5 tons and cracks were only observed on reaching a definite loading stage. Consequently the true load at the first crack might be as much as 5 tons smaller than the observed load and thus the ratios $\frac{P}{W}$ given are likely to be higher than their true values. For group C these ratios vary from 0.95 to 0.99, which shows that a softer concrete reduces the margin between the cracking load and the ultimate load. The ratios $\frac{P}{W}$ for groups D and E are generally within the range covered by group A.

In contrast to this the M.S. columns did not cra before the ultimate load was reached so that no warni of failures was given. The sudden failure of M. columns has been pointed out by previous investigato

Strains up to 0.003 were measured for most colum of low strength concrete, irrespective of the amounts reinforcement. The maximum observed strains we generally lower for columns of high strength concre but a strain of 0.0031 was measured for column (group C) before the ultimate load was develope Evans[2] pointed out that cylinders of low streng concrete fail when the strains have become many tim as great as those in cylinders of high strength.

Hajnal-Kónyi recorded strains of 0.00515, 0.005 and 0.0061 for columns with 7 per cent. steel in co cretes of 2750, 2760 and 6560 psi. respectively. The are nearly double the maximum strains observed in t present tests. Furthermore, columns in the lo strength concrete did not develop full strength befo strains of 0.005 were reached; Columns 8 and (group A) which had 8 per cent. steel in concretes 3130, and 3,100 psi. cube strength respectively, reach the ultimate load at a strain of 0.0025. It is believ that the high strains recorded in Hajnal-Kónyi's tes are due to the large amount of lateral reinforceme provided.

To obtain 7 per cent. reinforcement eight $\frac{3}{4}$ in. ba were used, symmetrically placed at the corners a sides of the columns. Two sets of links were need

Table 6

1 Group	2 Column	3 C_u (p.s.i.)	4 C_p (p.s.i.)	5 $A_c.C_p$ (tons)	6 C_s (p.s.i.)	7 $A_s.C_s$ (tons)	8 $W_1 = A_c.C_p + A_s.C_s$ (tons)	9 Observed Load W (tons)	10 $\frac{W_1}{W}$
A	1	6700	4800	305	54,000	37.6	342.6	350	1.0
	1a	6800	4900	311.5	54,000	37.6	349.1	350	1.0
	2	4930	3450	96.1	54,000	37.6	133.7	130	0.9
	2a	5500	3840	107	54,000	37.6	144.6	150	1.0
	3	5900	4200	114.1	54,000	73.9	188	185	0.9
	3a	4100	2810	76.5	54,000	73.9	150.4	142	0.9
	4	4670	3270	86	54,000	122	208	200(1)	0.9
	4a	3900	2670	70.3	54,000	122	192.3	196	1.0
	5	3660	2450	152.7	54,000	37.6	190.3	187	0.9
	5a	4400	3020	192	54,000	37.6	229.6	230	1.0
	6	2790	1850	51.6	54,000	37.6	89.2	95	1.0
	6a	2940	1950	54.3	54,000	37.6	92	97	1.0
	7	3400	2300	62.6	54,000	73.9	136.5	131	0.9
	7a	3260	2180	59.3	54,000	73.9	133.2	136	1.0
	8	3130	2100	55.3	54,000	122	177.3	178	1.0
	8a	3100	2080	54.6	54,000	122	176.6	172	0.9
B	1	5950	4200	267	40,000	27.8	294.8	280	0.9
	1a	5300	3720	236.5	40,000	27.8	264.3	260	0.9
	2	5300	3720	104	40,000	27.8	131.8	130	0.9
	2a	5600	3980	111	40,000	27.8	138.8	140	1.0
	3	4170	2860	77.7	38,000	54.7	132.4	135	1.0
	3a	5300	3720	102	38,000	54.7	156.7	160	1.0
C	1	6700	4800	307	55,000	38.3	345.3	325	0.9
	2	4700	3250	90.6	55,000	38.3	128.9	125	0.9
	3	4700	3250	88.6	55,000	75.2	163.8	174	1.0
	4	4540	3140	82.7	57,000	129	211.7	212.5	1.0
D	9	2690	1720	45.2	61,000	138	183.2	180	0.9
	10	4300	2940	80	54,000	75.3	155.3	154	0.9
	10a	5950	4200	117	54,000	75.3	192.3	200	1.0
E	2	5290	3700	103.2	58,000	40.5	143.7	150	1.0
	2a	5400	3780	105.5	58,000	40.5	146	154	1.0
	3	4930	3450	94	58,000	79.2	173.2	170	0.9

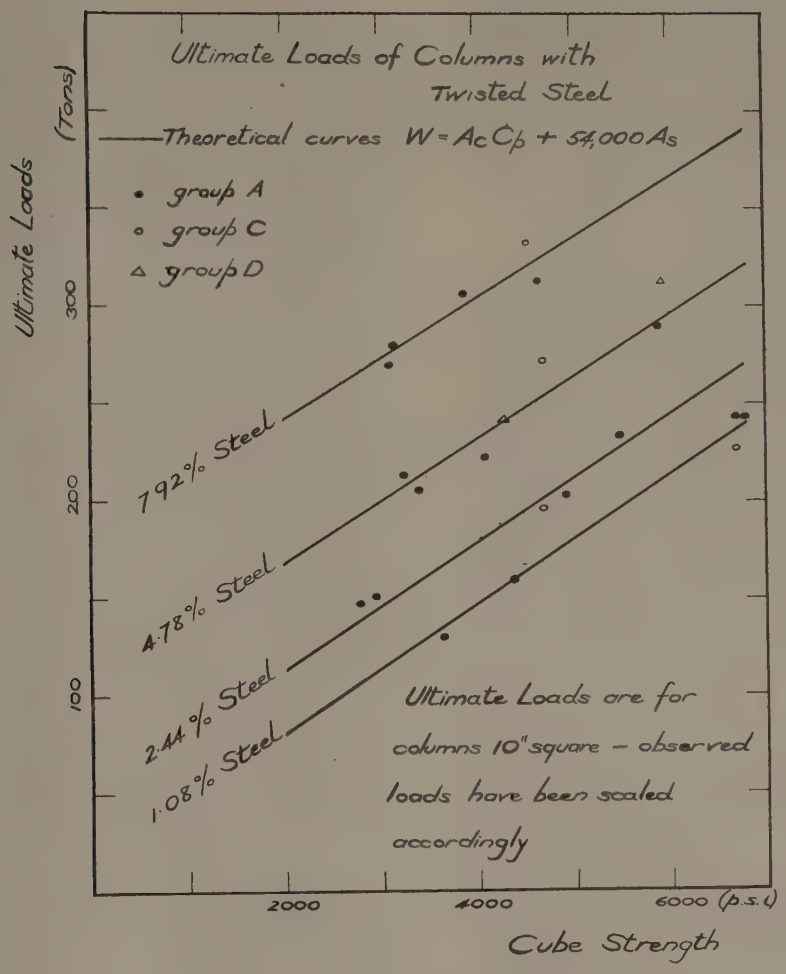

Fig. 9

o tie the vertical bars properly ; one set enclosed all ight vertical bars ; the other set enclosed only the our bars at the sides of the columns. Each set was rranged at 6 in. pitch and staggered, so that the pitch etween adjacent binders was 3 ins.

The Effect of Links

The functions of closely-spaced links in a column are o restrain the shearing of the concrete and to limit the uckling of the main bars. At normal spacings ties re relatively ineffective.

Columns of group B were reinforced with M.S. and ad links at 8 in. pitch. No cracks were observed efore the ultimate load was reached at a strain of .002 although the steel yielded much earlier at a strain f 0.0014. The steel by itself has virtually no resis- ance to buckling once it has yielded ; obviously the over concrete alone must be strong enough to prevent he steel from buckling between the links. Other tests [8 9 10] on columns with link spacings up to 8 ins. onfirm that the full yield strength of the steel can be eveloped.

Hognestad[8] has recently made a series of tests on oth axially loaded and eccentrically loaded columns, n which the pitch of links was 8 ins. and the cover to he main bars was 1 in. It was pointed out that the main bars buckled only when the cover concrete had spalled off, long after the steel had yielded at a stress f 44,000 psi.

It is therefore unlikely that steel buckling could account for the fact that columns of groups A and C failed at much lower strains than some of the columns tested for Hajnal-Kónyi. Moreover, the two types of column were hardly different as far as steel buckling is concerned. The buckling length of the bars was 8 ins. in the present series and the cover was $1\frac{1}{2}$ ins. thick, whereas the buckling length of the bars in Hajnal-Kónyi's tests was 6 ins. but the cover was 1 in. thick. The main function of the links at 3 in. pitch in Hajnal-Kónyi's tests must have been to prevent early failure of the concrete by shearing and crushing.

Tests on columns with closely spaced links have not been conclusive. Both King[3] and Mackey[4] have suggested that steel buckling will cause premature failure of columns with ties at normal spacings, although various tests mentioned disprove this. Steel buckling will only occur if the cover concrete is weak. Tests have shown that a cover of 1 in. is sufficient to prevent buckling ; the Code of Practice (C.P. 114) now suggests a cover of at least $1\frac{1}{2}$ ins.

With a large number of links shearing of the concrete will be prevented and the column will shorten until the cover concrete, being unsupported, weakens and spalls off. The longitudinal steel may then buckle between the links. In tests reported by King a higher load was reached after spalling and therefore the links must have been sufficiently close-spaced to prevent steel buckling without aid from the cover concrete.

In the discussion to King's paper Hajnal-Kónyi showed that the ultimate loads of the large columns agreed with the expression.

$$W = A_s Y + 0.7 A_I u + 1.45\ A_2 Y_2$$

where A_I = core area of concrete

A_2 = equivalent area of binders

Y_2 = yield point of binders

u = cube strength.

This formula is similar to the usual formula adopted for the ultimate strength of spirally reinforced columns [9 10]. It should apply to columns in which the added strength due to binders would be greater than the loss of strength due to spalling of the cover concrete and there is a minimum quantity of links which will provide this added strength. Mackey's tests showed that more than 2.5 per cent. of lateral reinforcement in the form of ties is needed in columns reinforced with mild steel. With T.S. columns the effect is different since *every* addition to the number of links above the minimum required by the Code will increase the ultimate strain of the column and thus higher steel stresses will be developed with a corresponding increase in the ultimate column load. Some further effects of links are evident from Hajnal-Kónyi's tests on columns with M.S. The arrangement of reinforcement in these columns was identical to that in the columns reinforced with T.S. Columns with links at 6 in. pitch failed without warning when the ultimate load was reached: maximum strains measured were 0.00163 and 0.00134 for low and high strength concretes. The columns with links at 3 ins. cracked at 97 per cent. and 96 per cent. of the ultimate load and the maximum strains observed were much higher, namely 0.00276 and 0.00193 respectively. The steel buckling length was the same in each column. Hajnal-Kónyi pointed out another effect of links in low strength concrete, namely that the maximum measured strain of a column with links only is substantially greater than that of prisms in plain concrete.

Columns with T.S. and M.S. Reinforcement

Tests have shown that for a normal tied column the useful stress in T.S. varies only from 54,000-57,000 psi. for widely differing amounts of reinforcement and concrete qualities. Some of the columns tested for Hajnal-Kónyi underwent large deformations before the failing load was reached but this was mainly due to the large number of links provided rather than the effect of α. Similar deformations were observed for columns 9 and 9a, which also had links at 3 in. pitch. A paper by F. Baraville[11] reports that the full proof stress of 'Torstahl' (67,500 psi.) can be utilised in a column if sufficient lateral reinforcement is provided, but unfortunately no test details were given. A suitable formula taking into account the effect of links would probably be in the form of equation 2 in which the steel stress is not a function of (α) alone, but of $\left(\frac{\alpha}{p^1}\right)$, where p^1 is some function of binder pitch. Steel stresses may be expected to vary from 54,000 psi. up to the proof stress, 60,000 psi. However tests to determine the effects of close-spaced links are hardly worthwhile since it is more economical in practice to keep the number of links to a minimum and to increase the amount of vertical steel if a stronger column is needed.

Comparing Figs. 4 and 5 shows the essential diff[erences] in the behaviour of M.S. and T.S. columns. M[.S.] steel yields at a strain of about 0.0013 and hence [no] further load on the column is carried by the concre[te]. The ultimate load is reached when the concrete develo[ps] its full strength, hence the strain at ultimate load [is] the same as for plain columns and no cracks are o[b]served before the maximum load is reached. Sin[ce] concretes of normal mixes usually reach maximu[m] strength at strains of about 0.0015 it follows that m[ost] columns with M.S. will develop the ultimate load [at] this strain. The Stuttgart tests confirm this. T[he] higher strains observed for columns of group B we[re] due to the highly compressible type of concrete.

Columns reinforced with T.S. differ inasmuch as t[he] load on the steel increases while the concrete yiel[ds] after developing full prism strength. For a concre[te] such as that in columns of group A the amount of yie[ld] is considerable. During this period the concrete crac[ks] and ample warning of failure is given.

Hajnal-Kónyi pointed out that the effect of creep [in] columns with T.S. is to increase the factor of safety, [in] contrast to columns with M.S. Creep causes a tra[n]ference of load from the concrete to the steel so th[at] under sustained loads the yield point of mild steel m[ay] be reached. However this does no harm since t[he] concrete is available to take up any additional loads [on] the structure. The investigations at Illinois show[ed] that sustained loading has no effect on the ultima[te] strength of columns reinforced with mild steel.

Creep may cause substantial deformations in [a] structure. Concrete cylinders loaded for $2\frac{1}{2}$ years [to] half the ultimate strength developed strains of 0.00[3]. If columns reinforced with T.S. developed such strai[ns] then the stress in the steel at ultimate load would [be] about 58,000 psi. whereas in a short laboratory te[st] the useful stress developed is 54,000 psi.

Design Stresses

The behaviour of T.S. as compression reinforceme[nt] is superior to M.S. both as regards the amount [of] deformation at ultimate load ("toughness") and warning of failure by cracking. The stress develop[ed] by T.S. is much higher than the yield stress of mild st[eel] even in a laboratory test of a few hours duratio[n]; creep due to sustained loads should allow even high[er] stresses to be developed.

The Code of Practice bases the working loads [for] axially-loaded columns on the expression $W = A_c C_p + A_s Y$ together with certain load factors on t[he] steel and concrete. A working stress of 18,000 psi. [or] half the guaranteed yield point up to a maximum [of] 20,000 psi. is allowed for mild steel. Mild steel m[ay] develop the yield strength, 36,000 psi., and therefo[re] the load factor on the steel is 2.0.

At present a stress of 20,000 psi. is allowed for T[.S.] but tests have shown that a useful stress of at le[ast] 54,000 psi. will be developed and so in order to ha[ve] a load factor of 2.0 on the steel a stress of 27,000 p[si.] should be adopted.

The working loads and safety factors for the colum[ns] tested have been calculated in Table 7. Design stres[ses] for the concrete are based on the suggestion of the Co[de], so that working stresses are equal to ($\frac{1}{3}$ x 0.76 x cu[be] strength). It has been shown that the ultimate co[n]crete stress is approximately ($\frac{2}{3}$ x cube strength) [or]

Table 7

Group	Column	Cube Strength (p.s.i.)	Allowable Concrete Stress (p.s.i.)	Concrete Load (tons)	Steel Load (tons)	Working Load (tons)	Ultimate Load (tons)	Load Factor
	1	6700	1690	107	18.8	125.8	350	2.78
	1a	6800	1720	109	18.8	127.8	350	2.74
	2	4930	1250	34.9	18.8	53.8	130	2.42
A	2a	5500	1390	38.8	18.8	57.6	150	2.60
	3	5900	1490	40.5	36.9	77.4	185	2.39
	3a	4100	1040	28.3	36.9	65.2	142	2.18
(T.S.)	4	4670	1180	31	61	92	200(1)	2.18
	4a	3900	986	25.9	61	86.9	196	2.26
	5	3660	927	59	18.8	77.8	187	2.40
	5a	4400	1115	70.9	18.8	99.7	230	2.30
	6	2790	707	19.7	18.8	38.5	95	2.47
	6a	2940	745	20.7	18.8	39.5	97	2.45
	7	3400	862	23.4	36.9	60.3	131	2.18
	7a	3260	826	22.4	36.9	59.3	136	2.3
	8	3130	792	20.8	61	81.8	178	2.18
	8a	3100	786	20.6	61	81.6	172	2.11
	1	6700	1690	107	18.8	125.8	325	2.58
	2	4700	1190	33.2	18.8	52.0	125	2.40
C	3	4700	1190	32.3	36.9	69.2	174	2.52
(T.S.)	4	4540	1150	30.2	61	91.2	212.5	2.33
	1	5950	1500	95	13.9	108.9	280	2.57
	1a	5300	1440	91	13.9	104.9	260	2.48
B	2	5300	1340	37.4	13.9	51.3	130	2.54
(M.S.)	2a	5600	1420	39.6	13.9	53.5	140	2.62
	3	4170	1060	28.6	27.4	56	135	2.41
	3a	5300	1340	36.4	27.4	63.8	160	2.50
	9	2690	691	17.9	61	78.9	180	2.28
D	10	4300	1009	27.4	37.6	65	154	2.37
10a	10a	5950	1507	40.9	37.6	78.5	200	2.55
	2	5290	1304	37.4	18.8	56.2	150	2.62
E	2a	5400	1368	38.1	18.8	56.9	154	2.70
	3	4930	1249	34	36.9	70.9	170	2.40

Note.—(1) Ultimate load not reached.

hat the load factor on the concrete is $\left(\frac{3}{0.76} \times 2/3\right)$ or 2.65.

Columns with T.S. steel develop their working loads at slightly higher strains than the columns with M.S. due to the higher working stresses, but the difference is negligible and could scarcely be detected from the load-strain curves. In any case creep would cause much higher strains to be developed in an actual structure, so that the effect is of no significance.

Conclusions

1. The ultimate loads of axially loaded columns reinforced with square twisted steel (T.S.) in which the pitch of binders and cover thickness complied with the minimum requirements of B.S.C.P. 114, agree satisfactorily with the expression

$$W = A_c C_p + A_s . C_s$$

where A_c = area of concrete

A_s = area of steel

C_p = strength of concrete as a plain column (which may be taken as $\frac{2}{3}$ cube strength)

C_s = steel stress of at least 54,000 psi.

2. In all cases columns with a concrete of normal stress/strain characteristics and reinforced with T.S. undergo about 50 per cent. greater deformation than columns with mild steel (M.S.) before the ultimate load is reached. At the same time ample warning of failure is given in columns reinforced with T.S. after the appearance of cracks in the concrete.

3. The effect of creep under sustained loads is to increase the factor of safety for columns reinforced with T.S. whereas it has no effect on columns reinforced with M.S.

4. The permissible steel stress in columns reinforced with T.S. steel can be increased from 20,000 psi. (the maximum value permitted at present in C.P. 114) to 27,000 psi. so that the load factors are similar to those allowed at present for columns reinforced with M.S.

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Research Work in Soil Mechanics at the University College of Swansea*

By A. A. Fordham, Ph.D., M.I.Struct.E., A.M.Inst.C.E.

The subject matter of this paper is intimately connected with the interests and activities of the Institution of Structural Engineers inasmuch as the problems in soil mechanics that we have studied in the Engineering Department of Swansea University College have originated in the panels, committees and sub-committees set up in London by the Institution.

The work to be described has been carried out by the author and by post-graduate research workers under his direction, over a period of 21 years since 1932, and may conveniently be considered as a contribution towards the solution of three main problems.

(1) Measurement of the active pressure and the passive resistance of granular soil.
(2) Measurement of the stability of pad foundations subject to overturning moment.
(3) Earth pressure on cantilever walls and tee-shaped walls.

In the course of this work several secondary matters have arisen, including

(a) The design and construction of pressure gauges of various types.
(b) A study of " at rest " pressure for no movement of the soil.
(c) Overturning of poles of circular and of rectangular section set into soil.
(d) The production of casts of soil disturbance due to movements of model walls, poles and pad foundations.
(e) A study of the double slip planes in the case of tee-shaped walls.
(f) Pressure of granular material in bins and silos.
(g) Flow lines of drainage behind walls with water-logged backing.

Almost the whole of the work has been carried out using sand obtained from local shore dunes which provide a clean granular material when dry, and a granular/cohesive material when moistened with water.

In these days when there are so many lectures, conferences and text books on soil mechanics it is useful to remember that the year 1932, according to " Capper and Cassie," marks the beginning of the serious study of soil mechanics in Britain with the publication of the work of Professor Charles F. Jenkin.

Turning now to the problems investigated, the first arose in 1932 when the author was invited to join a panel of the Institution under the chairmanship of Mr. Ernest Granter, to draw up a Report on Gravity Retaining Walls.

The terms of reference for the panel included th study of published information on the subject of acti and passive earth pressures together with the mo recent researches.

In order to check the suitability of the Wed Theory as a basis for calculation of earth pressures o retaining walls, an experimental model wall was co structed and tests carried out in the open air at th College playing fields. The wall consisted of a vertic steel plate cantilever ¼ in. thick, 2½ ft. high and 7½ f long set in a concrete floor at a distance of 6 ft. in fro of a bank of earth. Timbers placed between the wa and the bank at each end of the wall completed th test bin into which sand could be placed and withdraw Active pressure in magnitude and distribution wa calculated from the deflection curve of the cantileve whilst passive resistance was measured by pulling th top edge of the plate backward against the sand. Th results suggested that the Wedge Theory would giv results satisfactory for design and on the safe side.

In order to make an independent check upon th tests described above, a pressure gauge was designe and constructed to record the pressure of soil on plate 4 ins. square moving either away from or towar the soil. This gauge was set on a concrete foundatio in a fixed position in relation to a large timber test wa built in a sand pit at Pennard near Swansea. Record were made of the pressure intensity with variou depths of sand above the centre of the gauge. Th values obtained agreed with the results found by th first method. (*Engineering* 14.5.37 and 30.4.43).

Both of these series of experiments were carried ou under practical outdoor conditions and between the involved the shovelling by hand of sand equivalent t about 400 tons weight.

The second of the three main problems arose i 1940 when the Foundations Sectional Committee of th Institution appointed a Sub-Committee to draft Report on the Design of Foundations, under the chai manship of Dr. S. B. Hamilton. In 1943 the autho was invited to carry out experiments upon pad founda tions subject to overturning moment. This investiga tion was attempted by means of a 6-inch concrete cub full size foundation blocks being precluded owing t the magnitude of the forces involved.

Briefly, the method employed consisted in a com parison of the overturning moments required (A) wit the concrete block resting on the surface of a san foundation and (B) with the block sunk below th surface of the sand. The results were published i *The Structural Engineer* for July 1944 and reference made to them in Dr. Hamilton's paper in *The Structur Engineer* for September 1945.

* *Paper read before the Wales and Monmouthshire Branch of the Institution of Structural Engineers at Cardiff on October 28th, 1953, at Swansea on November 4th, 1953 and at Colwyn Bay on November 7th, 1953. Awarded Wales and Monmouthshire Branch Prize—1953-4.*

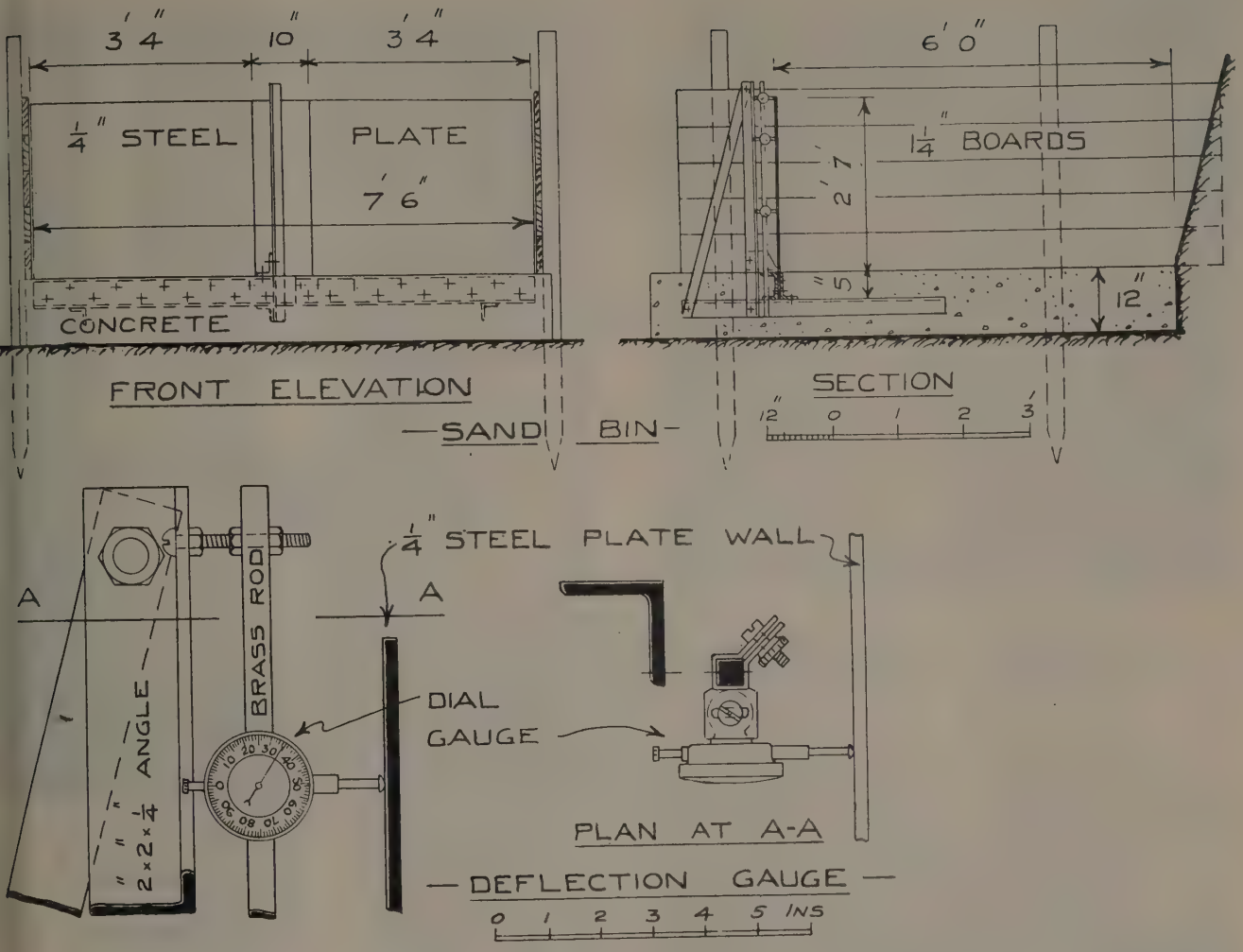


Fig. 1.—Apparatus for Sand Pressure Tests

Fig. 2.—Overturning of Model Foundation Blocks in Dry Sand. General View of Apparatus

Fig. 3.— Casts showing Slip Planes in Dry Sand due to overturning of Model Foundation Block

Fig. 4—Casts showing Slip Planes in Dry Sand due to overturning of Model Foundation Block

Since that preliminary work was done the subject has been very carefully studied by Haydn E. Evans using concrete model foundation blocks of various sizes.

In his experiments Dr. Evans employed a large rectangular metal tank 8 ft. long to contain sand. Two exactly similar concrete blocks were set in the sand at a suitable distance apart having vertical columns attached and a horizontal force was applied and measured to overturn them by pulling the columns together. This method was originally suggested to the

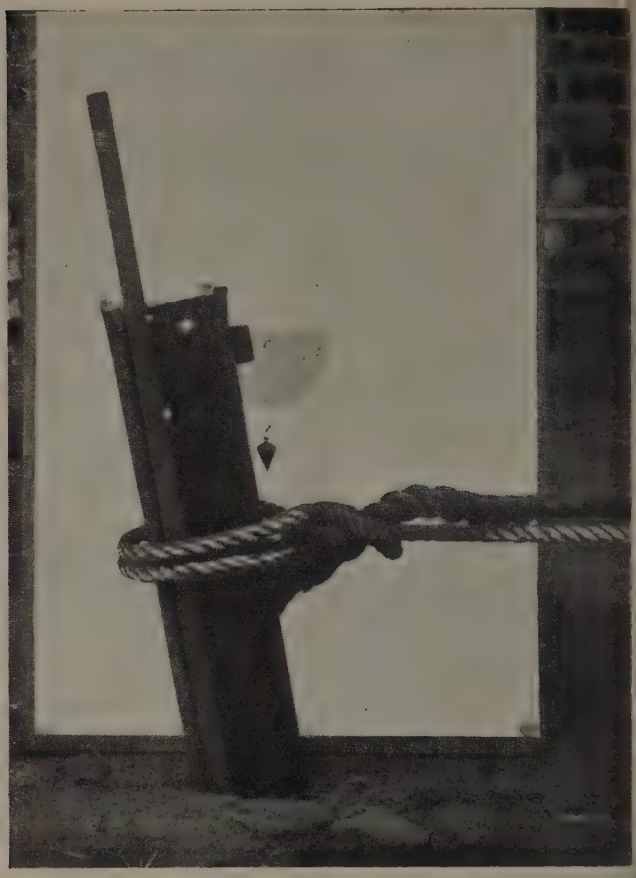


Fig. 5.—Overturning of 3 in. x 2 in. Rectangular Section Bar driven 12 in. deep in clay

author by Dr. T. E. N. Fargher when he was Technica Officer to the Institution. Dr. Evans obtained sati factory results but in a form too complicated fo application to a practical design problem. He hope to prepare a simplified method of applying thes results and to publish this work in the near futur

Arising out of the work on the overturning o foundation blocks the resistance to overturning o poles presented a subject to be investigated. Exper ments on the overturning of rectangular section mod poles led to the suggestion of the quadrant theor described by the Author in *The Structural Engine* for July 1944. This method of analysis is supporte by some experiments carried out in 1947 upon the sit of the extensions to the College Engineering Depar ment. A rectangular-section steel bar was driven int the clay soil and overturned by a horizontal pu applied some distance above ground by a wire rop from a winch and tripod. When the bar was com pletely overturned the disturbed soil was carefull removed and the hole filled with wet concrete. Afte setting, the concrete was dug out and provided a cas of the disturbed soil.

In the case of poles of circular section the form of th soil disturbance is more complex. This has bee studied at Swansea College by T. E. H. Williams, no of King's College, Newcastle-upon-Tyne. To D Williams belongs the credit for the idea of making soli casts showing soil movements when a model pole overturned. For this purpose he used dry granula material in the form of plasterer's cement added t sand and to powdered anthracite giving horizonta laminations in which slip planes were developed b

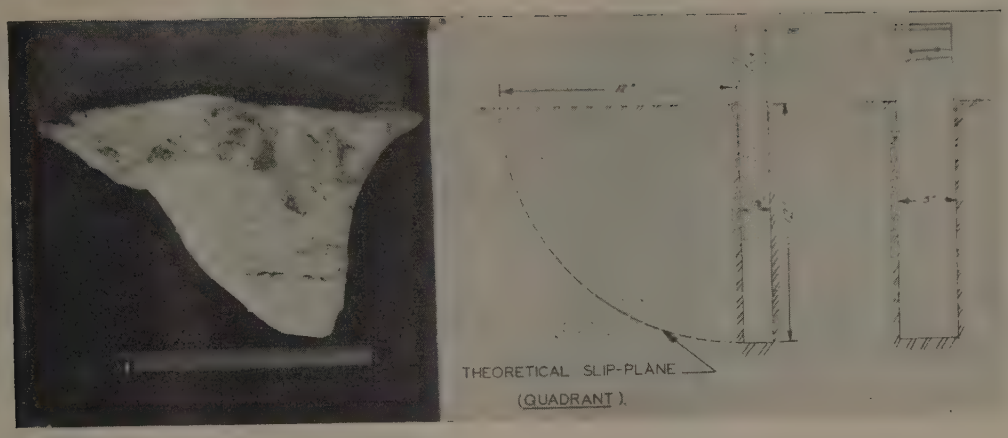


Fig. 6.—Cast of Disturbed Ground compared with Theoretical Quadrant

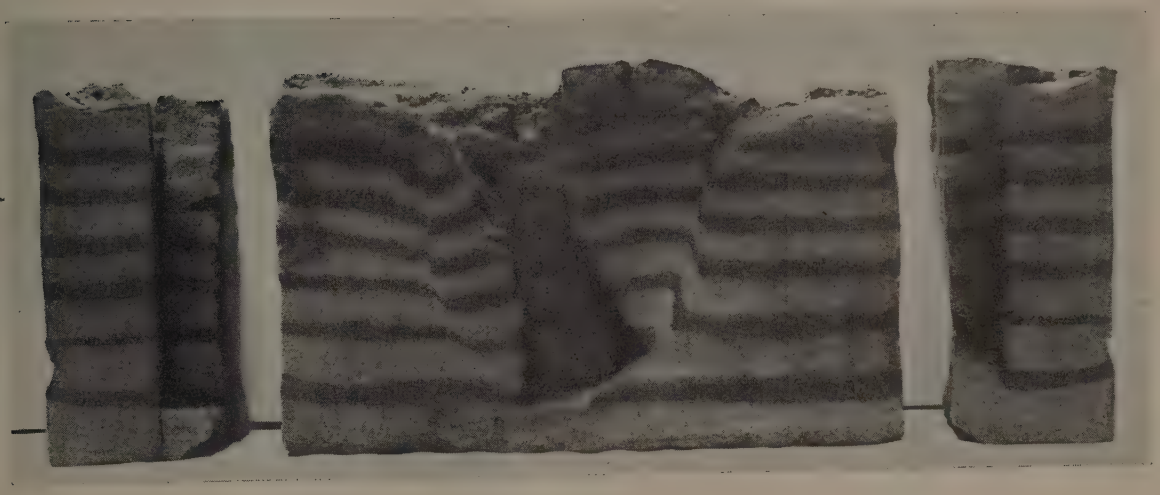


Fig. 7.—Cast showing Soil Disturbance in Dry Sand due to overturning of a Rectangular Section Model Pole

overturning the pole set therein. Using " teepol " as a wetting agent it is possible to solidify the dry material with water and after the mortar has set sections may be cut in any direction to show the precise nature of the internal movements. This process was later improved by the use of a lean mix of portland cement and sand with cementone to produce strata of various colours for the detection of slip planes. In this form the method has been invaluable in subsequent research work.

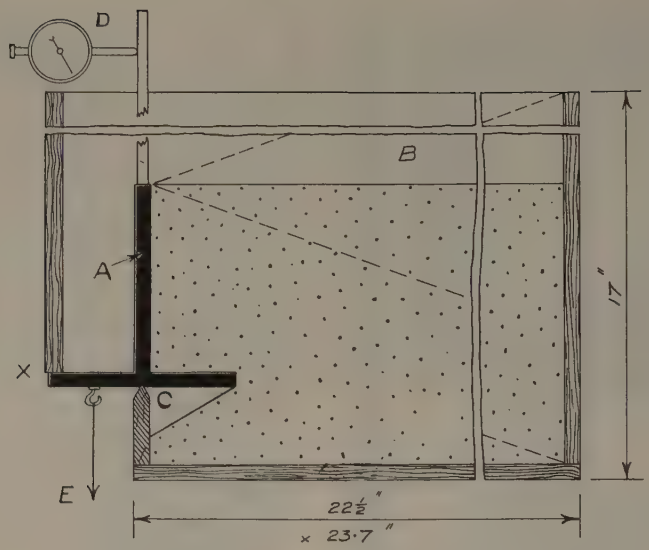
In passing, it is interesting to recall that " teepol " was used as a wetting agent to eliminate segregation in the concrete of Claerwen Dam for Birmingham Waterworks.

Dr. Williams has written of his experiments in an article published in *Engineering* Feb. 1952.

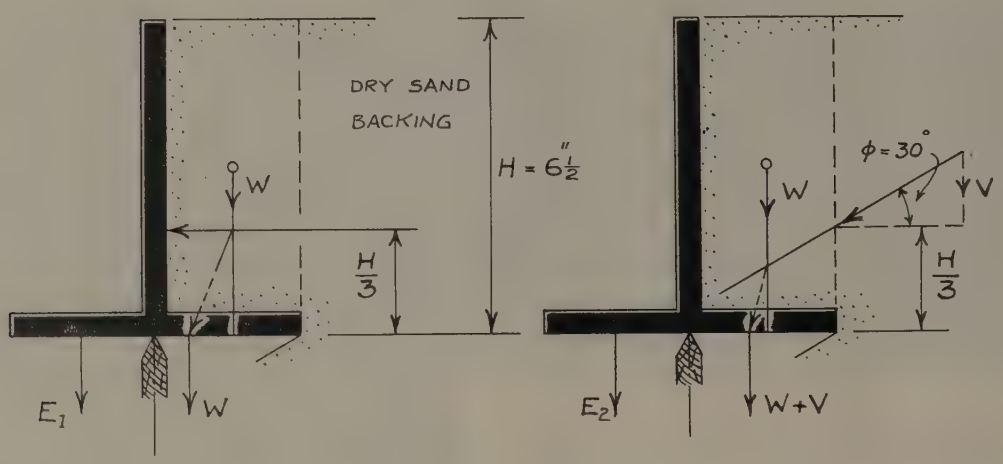
Coming now to the third and last of the three main problems, the experiments on model tee-shaped walls originated in 1945 in connection with the preparation of the Code of Practice on Earth Retaining Structures. Mr. Wallace A. Evans, M.I.Struct.E. and the author were appointed by the Institution Panel X(1) dealing with Fundamentals, to form a sub-panel and report upon cantilever Walls and Stepped Walls.

As a result of this and in order to obtain information on the behaviour of tee-shaped walls, a timber model was constructed and tests carried out to determine whether the ordinary wedge theory formulae could suitably be used for estimation of the active earth pressure and its point of application. The model was an inverted tee section 2 ft. long about 6 ins. high x 6 ins. x $\frac{1}{2}$ in. supported on a longitudinal knife-edge of timber with suitable grooves in the base of the wall. A bin formed behind the wall contained the sand backing and the overturning moment was actually determined by the simple method of balancing with a measured force at a known distance from the fulcrum. As a result of these tests it was found that initial rotation corresponded with a horizontal pressure on a vertical plane at the heel whilst collapse occurred with a pressure on this plane inclined at the angle of internal friction of the sand, the pressures being as calculated by Wedge Theory. These results are incorporated in the Code.

This research was continued by two other post-graduate workers using an Aluminium Tee Section for the wall and an iron bin in place of timber. Arthur Evans, B.Sc., conducted tests similar to those already



Model T-shaped wall with no surcharge.



Figs. 8 & 9.—Preliminary Tests on overturning of Tee-shaped Wall. Comparison of Test Results with Wedge Theory

described for wall failure by tilting and found that two slip planes were developed in the sand movement behind the wall. Unfortunately he died before his results were complete.

The work was carried on by Bernard D. Vaughan, M.Sc., who modified the apparatus to study the effect of a horizontal movement of the aluminium tee-wall away from or towards the sand backing. He also obtained valuable information on the form of the double slip-planes and compared his findings with the theory put forward by Professor Jenkin and the tables of the Revised Wedge Theory.

Important amongst the secondary matters of research has been the measurement of the "at rest" pressure of soil. This value for no movement of the soil in relation to a structure must lie somewhere between the active and passive magnitudes. John Newton using a very sensitive spring-action pressure gauge of his own design, determined the coefficient of pressure at rest for the dry sand and was able to check his findings by another method. Dr. Newton also studied the lateral pressure of sand and of wheat on model bins, comparing his results with values calculated by Janssen's theory.

The passive resistance of soil is an important factor in estimating the resistance of foundation blocks to overturning and a study of it formed a preliminary to that research. By measurement of the force and at the same time obtaining the actual slip-plane for passive resistance, H. E. Evans was able, with a model wall constrained to move horizontally, to demonstrate the

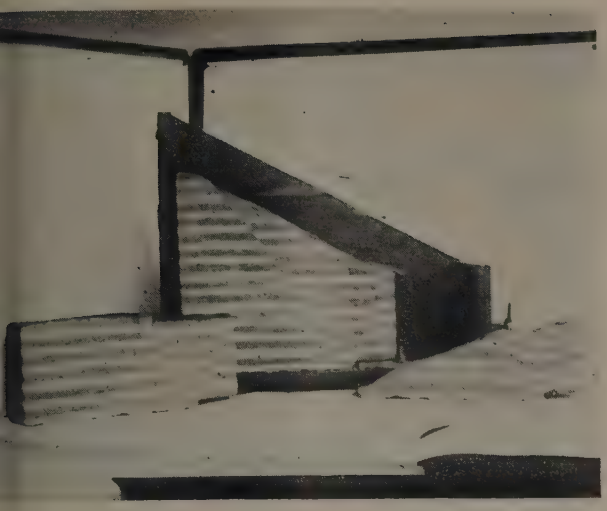


Fig. 10.—Double Slip Planes in Casts for Model Tee-shaped Wall moved away from Dry Sand Backing

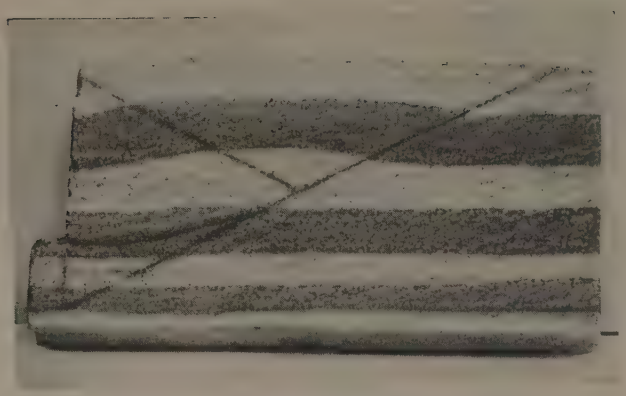


Fig. 11.—Cast showing Slip Planes of Resistance for a Model Wall moving horizontally against Dry Sand

general reasonableness of the logarithmic-spiral method of analysis.

The most recent line of research has just been completed by H. Clapham, M.Sc., who has studied the effect of weepholes and filter drains upon the pressure of waterlogged sand behind a model wall. This work has involved a study of experimental flow lines and furnished data for the location of drainage for maximum efficiency.

In the research work carried out during the past 5 years help and encouragement has been given to our workers by the Department of Scientific and Industrial Research, and thanks are especially due to Dr. Cooling for his kindness to three of our men who have spent short periods in his laboratories at the Building Research Station, Garston.

The constant interest of Professor Kastner in the work has resulted in the equipment of the Engineering Department at Swansea with standard soil testing apparatus without which the work would have been impossible; whilst he has encouraged the research workers to make their own special apparatus in the College Workshops with the assistance of the Steward Mr. Cunningham and his staff.

In conclusion I would like to draw attention to the fact, already mentioned, that the work described has sprung from and been stimulated by the activities of the Institution in London and represents an interesting example of its influence.

Book Reviews

Mechanics of Materials, 3rd Edition, by P. G. Laurson and W. J. Cox. (New York: John Wiley, 1954; London: Chapman and Hall.) 9 in. × 6 in. 414 plus vii pp. Price 46s.

This popular book on Strength of Materials, first published more than fifteen years ago, has now reached its third edition. This has been largely rewritten and tables have been revised to conform to the present American Standards and allowable stresses are based on current specifications. A minimum course in strength of materials is covered by the first thirteen chapters, the following seven chapters dealing with columns with eccentric loads, combined stresses, stresses produced by moving bodies, additional beam topics and eccentrically loaded connections. Frameworks are not included.

The text contains a large number of problems, many based on actual engineering structures. Many new problems and worked examples have been added to the new edition. The final chapter of the book consists of a group of "Comprehensive Problems," the solution of which requires the application of a number of principles drawn from different parts of the text.

Architects', Builders', Civil and Highway Engineers' Reference Book. General Editor, E. Molloy. (London: Newnes, 1952.), xii + 971 pp., 11 in. × 8½ in. Price £5 5s.

The title and scope of the 4th Edition of this important reference book have been extended to cover the subject of Highway Engineering, previously dealt with in a separate volume.

The subject matter has been divided into twenty-five sections, including chapters on surveys and site investigation, basic building and road-making materials, structural steelwork and piling, reinforced concrete, bridges, heat and sound insulation, contractors' plant and equipment, heating, ventilating and air conditioning. There is also a progress and development section, which gives technical notes on new materials, forms of construction and equipment which have recently been placed on the market.

Special articles contributed by experts are on school buildings, water resources of Great Britain, and Highway Legislation.

The volume is well-produced and contains a directory of manufacturers and a useful index.

(*continued on page 364*)

Colliery Structures*

Discussion on the Paper by Mr. G. P. Bridges, M.I.C.E., M.I.Struct.E.

THE CHAIRMAN introduced the Lecturer, who then presented his paper and showed a number of illustrations.

The Chairman proposed a vote of thanks to Mr. Bridges for his interesting paper, and this was carried with acclamation.

Discussion

Mr. F. G. GLOSSOP (National Coal Board), opening the discussion, said he was particularly interested in Mr. Bridges' paper because some of the structures to which the Author had referred were built in the North Western Division and he was aware of the foundation difficulties which had arisen. One of the headgears illustrated was built on a mixture of sea-sand and silt in proximity to the River Dee, and there it was necessary to put down a particular sort of foundation which Mr. Bridges might describe for the interest of the members.

The Author had included in one of his later slides a compressor house where there had been great difficulty with the site because it was on peat. In fact the railway sidings had actually got down into the peat bed.

With regard to the paper in general, Mr. Glossop said as a customer, if he might so put it, there were one or two points he wished to make. Generally, reinforced concrete structures were preferred in the North-west, largely because of the absence of corrosion, the relative ease of maintenance and the absence of vibration. That applied especially to the tower-mounted winders to which Mr. Bridges had referred. Some of the headgears were of noticeable height—say 150 to 200 feet—and it was obviously necessary for them to be stable because the shafts beneath might be anything up to 3,000 feet in depth, and one could not afford to have the tower rocking about otherwise the cages would not run as truly and smoothly as they ought. But again, speaking from the point of view of the customer, he would like something which could be erected more quickly. In some of the cases illustrated the time for erection had been four or five months, certainly in some of them there had been attendant difficulties due to the shaft and the old headgear being in existence beneath, and the fact that coal winding operations had to be maintained, but at the same time he wondered if structural engineers could assist in getting newer, different forms of construction which would enable the job to be done more quickly. These new collieries were costing millions of pounds and it was obvious that time must be a very important element, and anything that the structural engineer could do to expedite the erection of the works would be very much appreciated by the mining engineer.

Mr. Glossop emphasised the rapidity with which technique was changing in mining, which indicated progress but prevented repetition work.

With regard to the dirt tips and the disposal of waste he would make no apology : it could be done if people were prepared to pay the price. Concluding his remarks, Mr. Glossop expressed his appreciation of the privilege which had been extended to him of being present that evening.

Mr. G. CUTTLE (National Coal Board) complimented the Author on a most timely and interesting paper—timely because the solution to the problem of economic winding tower design was one of the most pressing in so far as colliery surface structures were concerned. It was the policy and the trend today to instal tower mounted multi-rope friction winders where new winding plant was required, except in those instances where the possibility of utilising existing headgears or winder houses would more than offset the advantages otherwise to be gained. The adoption of multi-rope winding, apart from the many other advantages which accrued, could result, as Mr. Bridges had pointed out, in considerable saving in capital cost when compared with either single rope ground-mounted or single rope tower-mounted installations. That was mainly because the use of multi-ropes enabled the diameter of the winding drum to be reduced to about one-half that which would be required for a single-rope, with resultant reduction in overall dimensions and in weight in the winding equipment, and thus in the building required to house it. The speaker then showed a slide (Fig. A) illustrating some comparable installations based on equivalent winding duties, the first being a ground mounted installation designed with single-rope winders mounted on the ground, with four cages in the shaft and the second being the same installation but with two ropes instead of single ropes. Mr. Cuttle pointed out that even going to two ropes had reduced to some extent the height of the headgear because the pulley diameters were less, and because the diameter of the friction drum was less the size of the winder house had been appreciably reduced, but if the winders were put in the tower, and if the two-rope winders were in the tower there was a still bigger and more marked difference between the extent of the structures involved. When one came to four ropes there was a more marked difference still, and then if one changed from four cages to two cages, and to two cages and counter weight winding, one got a smaller structure still. The slide brought out the point made by the Author as to the savings in capital cost which could accrue by adopting the new principles.

To reap the full advantages, however, it was essential that the towers should be economically designed. That meant that the overall dimensions had to be kept down to a minimum and the structure must not be elaborate. Considerable thought had been given to the question of tower layout and tower design and the National Coal Board had evolved what they considered to be a reason-

* *Read before the Institution of Structural Engineers at* 11, *Upper Belgrave Street, London, S.W.*1., *on the* 28*th April,* 1955. *Dr. S. B. Hamilton, M.Sc., B.Sc.*(*Eng.*), *A.R.C.S., M.I.C.E.* (*President*) *in the Chair. Published in* THE STRUCTURAL ENGINEER, *Vol. XXXIII, No.* 4, *pp.* 111-9 (*April,* 1955).

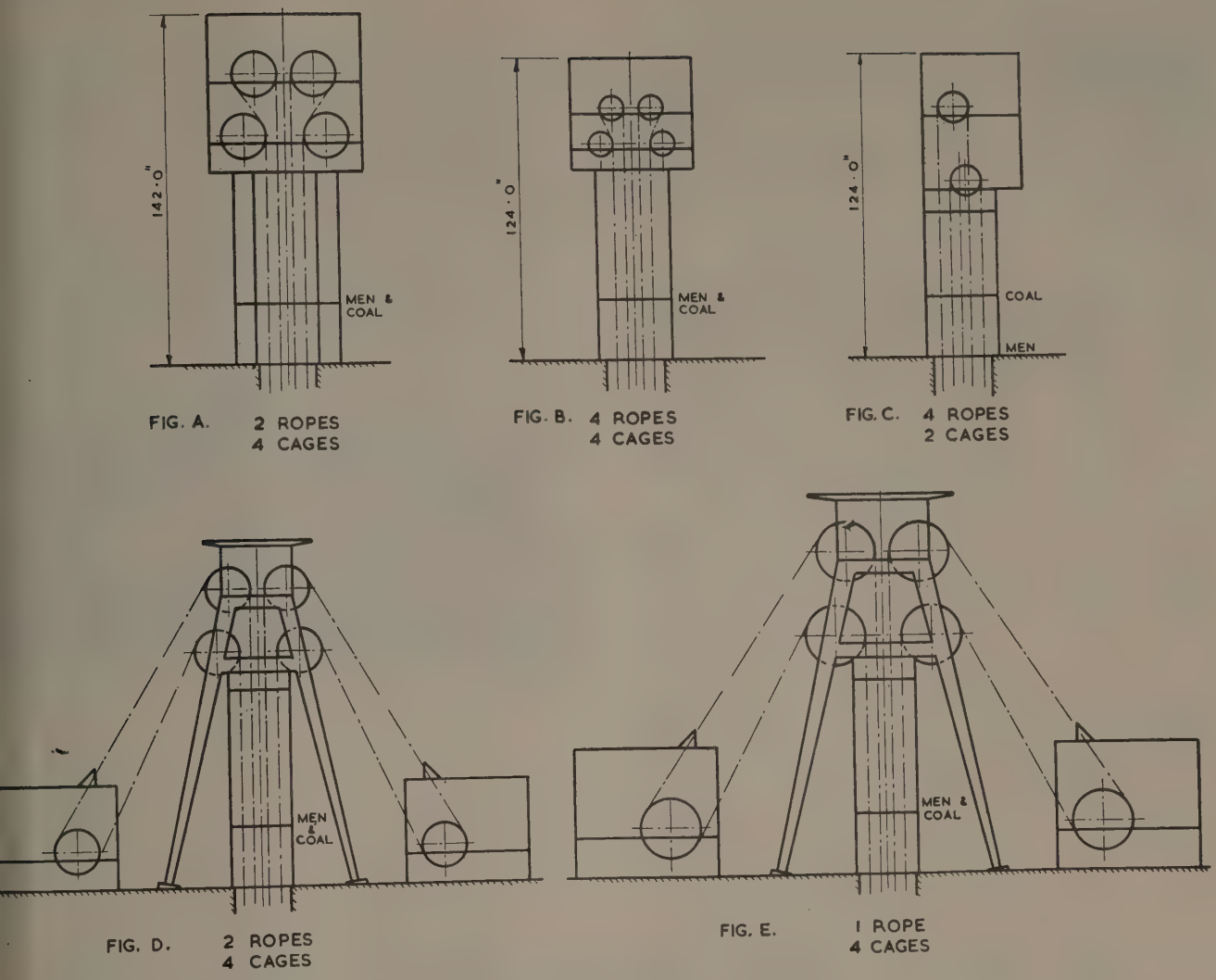


Fig. A.—Comparative Winding Arrangement

ble standard type of layout which would suit a very arge number of their new installations.

Mr. Cuttle then showed a slide (Fig. B) illustrating typical arrangement of winding tower, and said the najority of the new installations of 24 ft. diameter hafts would be equipped with two winding engines, robably each operating with a single skip or a single age and counter weight. The slide also indicated the inimum dimensions which could be achieved with eared type winders, and if they were to go to direct oupled winders it was possible that one dimension ould be reduced.

There was a lifting hatch provided on one side of the inder house, but the main tower structure was carried n columns at the corners, and the overhanging portion as screened and masked by a lift shaft which came up the corner. It had been felt that it would be prefer-ble and more economical for it to be an open type tructure, but if there was need for the shaft to be ncased, such as with upcast shafts for airlocking pur-oses, then that airlock would be carried from the guide tructure which generally surrounded the shaft, with ufficient clearance to allow for access for reasonable maintenance. It was felt that the buffer beams pro-vided to arrest the cages in the event of overwind, and any other retarding devices which had to be provided, should be carried from the headgear structure itself and not on the steel framing which was normally provided to carry the guides for the conveyances.

The tower designed to carry two winding engines with a geared type of drive measured about 44 ft. 6 ins. and the span across the base of the tower was about 50 ft. In the case of a skip installation the height might be increased according to the height of the skip discharge position.

With regard to winder drives it was generally proposed to adopt either direct coupled motors with overhung armatures, so that the drum shaft was supported only on two bearings or flexible drives through a unit gearbox. That meant that no shaft would be supported on more than two bearings and thus the bearing or gear alignment would not be affected by a slight deflection in the beams supporting the winding engine. There was one school of thought which considered that the tower could be made sufficiently rigid to preclude any possibility of deflection of the beams support in the winder, thus eliminating the need for the flexible gearing drive arrangement. It would be interesting to hear whether the Author considered this would be economical or whether he considered that the provision of a flexible geared drive or a two-bearing direct coupled arrangement would be a sound insurance policy.

In conclusion, mention had been made of the use of towers as sinking headgears. Generally on new sinkings it was the Board's intention to erect a tower up to the bumper beam level and to install the pulleys necessary for sinking. The tower could then be completed whilst the sinking proceeded. This simplified

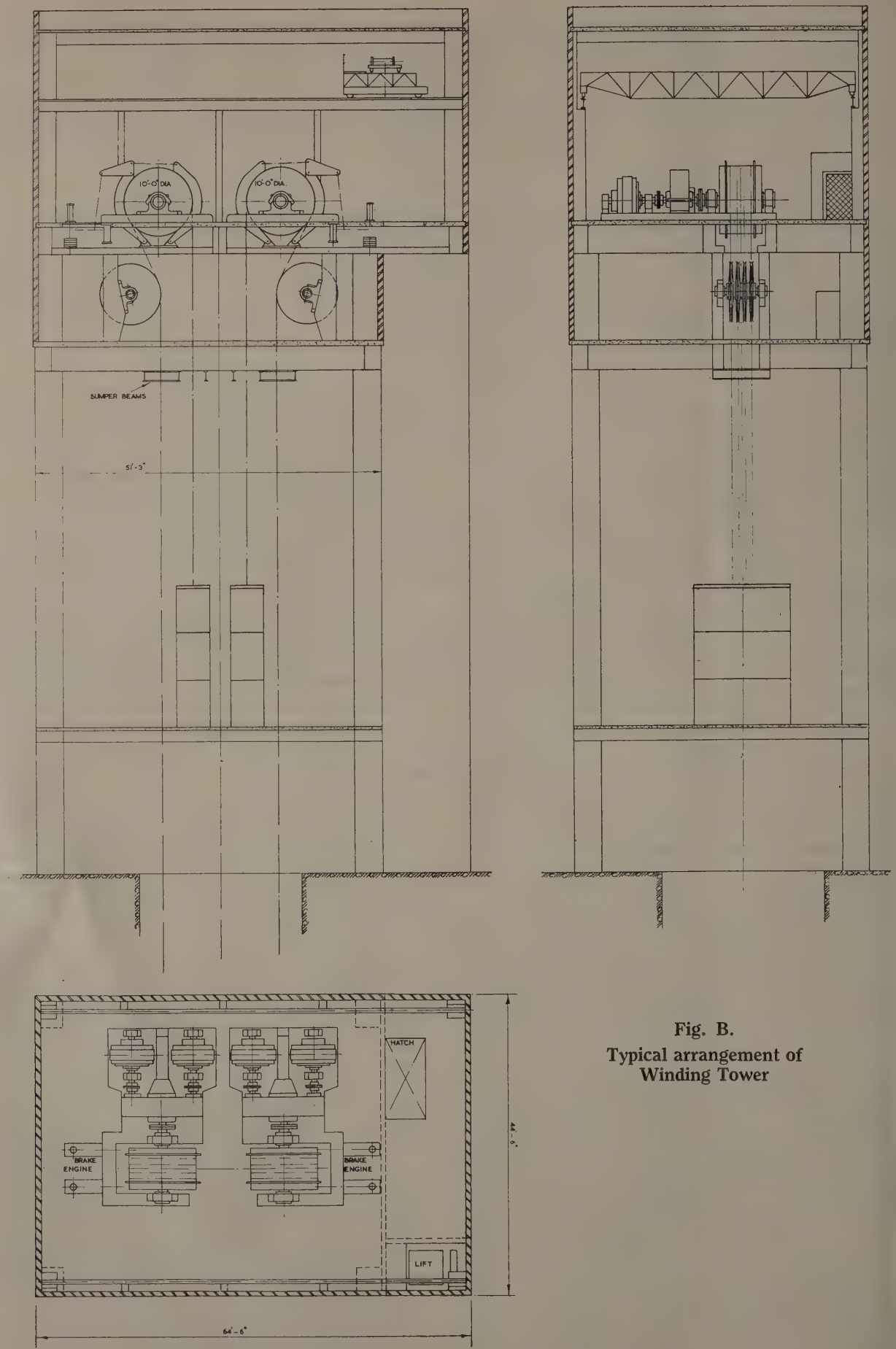


Fig. B.
Typical arrangement of Winding Tower

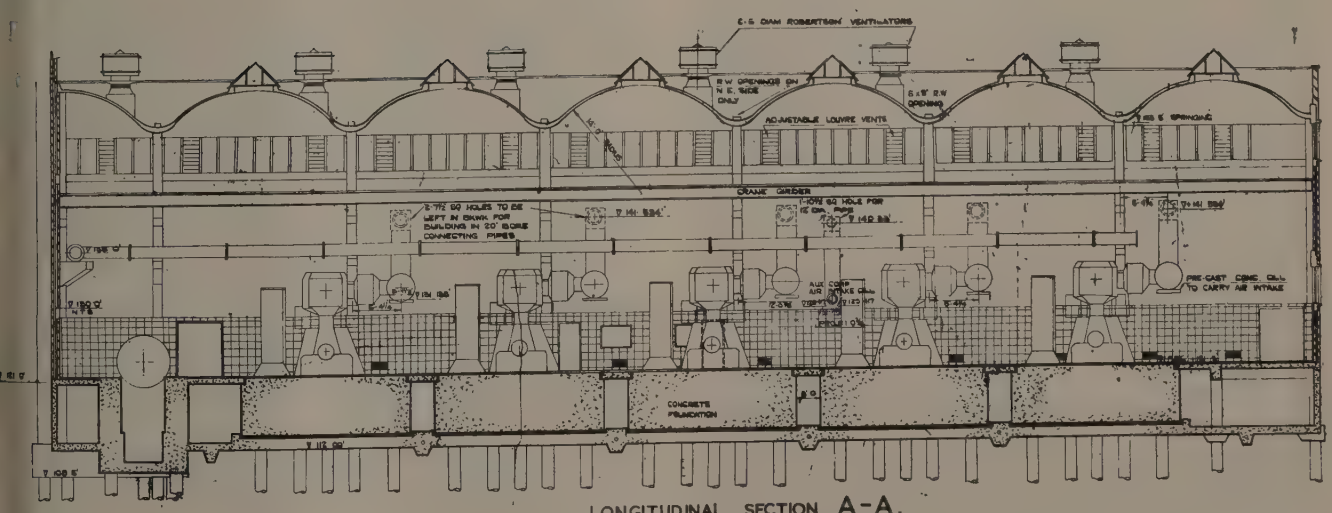
Fig. 7

Mr. A. P. Mason (Member of Council) said Mr. Bridges had performed a very valuable service in bringing before members and those present such a broad survey of the structures which were built in connection with collieries. There was a remark in the written paper, however, to the effect that it was only in recent years that the Koepe system had been used in Britain, and Mr. Mason said that had immediately recalled to him the fact that he had a lantern slide of one that was built in 1922 or 1923.

Mr. Mason then showed a slide of Murton Colliery, County Durham, which was designed by one of his predecessors and in those days, Mr. Mason said, he was too young to be entrusted with any part of it although he had been informed during the evening that a paper was written about that particular tower some years after it was built.

A second slide showed the Koepe winder tower at Bradford Colliery, Manchester, which was about 195 ft. high above ground level and was of the ground mounted type. The winding engine house was shown, and Mr. Mason said that the winder was driven by a 3,720 h.p. motor designed to raise 400 tons of coal an hour from about 1,000 yards depth. The tower was a skip winding type.

The next slide showed two Keope winder towers at Rothes Colliery in Scotland. Both towers were about 200 ft. high and the winders were mounted in the top of the tower instead of on the ground as in the previous illustration. Although both those jobs had only just



LONGITUDINAL SECTION A-A.

Fig. 8

he tower construction because it obviated the diffi-ulties of erecting around the temporary sinking head-gear and was economical because a separate sinking headgear was not required. There were, however, ases in water-bearing strata where the initial depth of new sinking had to be frozen. In such cases it was generally considered desirable to erect a temporary inking headgear and only to construct the permanent ower when the ground had been thawed out, since it had been found that considerable ground movement ook place during the process, and it would be inter-sting to have the Author's views on that matter.

been completed they were already out of date, so rapid was the march of progress. However, probably noth-ing more recent than those could be shown.

Mr. Mason also showed two further slides—the first being a drawing showing two sections of the tower, and the second showing the main beam, about 18 ft. deep, which ran across the top of the tower and which carried the gear.

Mr. H. C. Husband (Hon. Librarian) said it was a very great compliment to Mr. Bridges that his paper had aroused so much interest, but he had been reminded

of the very brilliant speech which was made the previous night—and which he had heard on the wireless—at the Royal Academy Dinner. One of Her Majesty's judges made this speech and, presumably demonstrating the clarity of the law, he quoted from the 1940 Finance Act which said something like "A long lease is a lease which is not a short lease." He had connected that expression with the design of structures for the National Coal Board because both Mr. Bridges and Mr. Glossop had talked about the majority of the buildings being temporary structures or structures with a short life. Quite clearly many of the ones which had been shown, including the very interesting 1922 tower, were by no means buildings with a short life. The only one which had been shown which could in any way be called a temporary building was an all-steel framed coal preparation plant which the Author had shown as an example of a plant which could be moved to another site. Had that in fact ever been done? He wondered whether the collieries had ever moved coal preparation plants from one colliery to another?

Arising out of the same point which Mr. Glossop had referred to was the possibility, or perhaps the desirability, of designing the buildings for coal preparation plants, which seemed to be getting bigger year by year, as more general purpose coal preparation plant buildings than tailor-made ones. In his view that would be possible, and it was a matter upon which the structural engineer could be of assistance. If only sufficient imagination could be used in the first place a fairly general purpose building could be built, much on the lines of some of those which had been shown by Mr. Bridges, with the lower part in heavy reinforced concrete. The bottom part with the railway tracks was not likely to be altered very much, but above that there could be a steel frame, not necessarily heavy enough to carry all the heavier items of plant, because those could be taken down to the concrete platform and, of course, when different types of plant were used in the future then the only thing that need be altered were the supports. There was nothing very new in that suggestion and in fact it had been carried out, but in his view it was better than to design a building to fit too closely to the machinery as a tailor-made job.

Another very interesting point had been raised about the very big motors. Obviously if one had to have a heavy high level shaft carrying a Koepe wheel one could not have too much standardization because one of the major factors influencing the economical design of the towers was the depth of the pit and the weight of the winding ropes and skips. It was a very nice thing to be able to support the motor shaft on two main bearings so that one simplified bearing alignment and bending moments on the shaft, but that led to another interesting problem which probably had something to do with the very big girder shown on one of the slides. The motor armatures were on the overhanging ends of the shaft and there was then a serious dynamic effect due to the possible variation in the air gap between those big armatures and the stators surrounding them. Obviously if the stators were on the opposite sides of the winding ropes and the pulleys they were mainly, if not wholly, connected by the structural frame of the building. In some of the designs he had been asked to examine, Mr. Husband said the tendency seemed to be to put in tremendous members in order to be very much on the safe side, rather than for the mechanical and electrical and, of course, the structural engineers to get together and really to determine the nature and the magnitude of the forces and the possible effect on the structure in the way of vibration, and the effect on the operating conditions of the motor itself. All those matters were with proper collaboration, capable of quite economic solutions. The Author himself had summarised the matter when he said that one must be careful not to have the members too thin otherwise they might vibrate, but a lot was known about designing frameworks which did not vibrate, or at least which had natural frequencies outside the range of vibration which might be applied to them.

Lastly, there was the question of the relative advantages of steel and concrete and the cost of maintenance which was wrapped up in the matter of temporary or permanent structures, but the steel work and concrete had a lot in common. A framework in concrete behaved in a very similar manner to a framework in steel: it was usually heavier, and that weight could be taken advantage of in the control of vibration; but his own opinion was that a structural engineer who knew his job could usually design a steel framework, just as he could design a concrete framework, to meet the particularly interesting conditions which were imposed by winding engineers.

Mr. Geoffrey Palmer (Member) said Mr. Bridge was a very old friend of his but they had not met for nearly thirty years until that evening. About thirty-five years ago they had attended lectures together when they were learning the elements of their job and since that time their ways had diverged—the Author, apparently, into reinforced concrete of a specialist nature, and his own mainly into steel work but also in recent years to a greater extent into reinforced concrete. The Secretary's remark before the meeting about the membership of the Institution now being 7,004 had reminded him of one of his larger jobs before the war—the largest theatre in Europe, where the seating was the figures reversed, namely 4,007.

Mr. Palmer said he knew nothing about colliery plant but there were one or two points in the paper which stood out. He thought the National Coal Board ought to make up its mind whether it wanted temporary buildings or whether it wanted permanent buildings, because if they wanted a permanent building then reinforced concrete might be very well, but if they wanted a temporary building then there was nothing whatever to beat steel work, which could be taken down and moved about. Reinforced concrete buildings had a great virtue in so far as they were heavier, as the last speaker had mentioned, and monolithic to a great extent and therefore more resistant to vibration, but there was no reason why a steel-framed structure should not also be monolithic to a great extent. It could be welded or it could be arranged so that the joints were in special places, and there was a great deal which the steel man could learn from the concrete man about continuity and stiffness.

One other point which had puzzled him a little was that he gathered that the latest idea of colliery winding gear was exactly what the lift engineers had been doing for very many years, namely a cage and a balance weight, and he wondered why it had not been done before. Was there any real reason why it was not done years ago? Again, it seemed that the size of the head-gear was largely determined by the number of ropes: with one rope one wanted a pulley of a certain size for it to go round, if it was divided into two ropes one wanted a smaller pulley, and if it was divided into four ropes one wanted a still smaller pulley; but why had that not been done before? Possibly Mr. Bridges would be able to say.

Mr. S. K. Liszka (Associate-Member) said the paper by Mr. Bridges, describing the conglomeration of independent structures which, working as a unit, produced coal, presented an interesting picture of the application of reinforced concrete in the coal industry. As he was interested in the design of colliery structures he wished to bring up a few points regarding the design of headgears. Firstly, the mechanical differences between Koepe winders and drum winders were well described in the paper, but the question was what really is their difference when it comes to designing the structures ?

Secondly, in the paper the Author had stated that "it is important that all members of a head frame of that type (downcast) should be designed to withstand shock and vibration which was being applied to it constantly at every wind." It would be interesting to know how Mr. Bridges includes the effect of vibration in his calculations for the reinforced concrete design.

Finally, Mr. Liszka said he would like to refer to the problem of stiffness and shock loading. His interpretation of the statements in the paper on that subject seemed to indicate that there was a contradiction : on the one hand it was said that all structural members should have a low stiffness ratio, and on the other hand it was claimed that reinforced concrete structures have a big advantage in resisting the stresses due to shock loading because of their natural stiffness. He would be pleased if Mr. Bridges would resolve that matter for him and tell him whether a structure with stiff joints and stiff members was more suitable than a structure with stiff joints and slender members to resist the shock loading.

Professor J. C. Mitcheson (Royal School of Mines) said he had listened to Mr. Bridges with very great interest and pleasure. They had first met in 1931 or 1932 in the depths of the Industrial Depression, which it would be remembered was particularly deep as regards the coal industry, which was then experiencing a terrible time with regard to its markets. Something had to be done to dispose of the coal, and in order to try and make it a little more attractive than that produced by their neighbours it had been decided to instal a modern washery. At that time money was extremely scarce and he could remember trying to persuade Mr. Bridges—he did not know him so well then—to cheapen the building in various ways. It was only afterwards, Professor Mitcheson said, when he had to deal with it that he was so thankful to have that roomy and well-designed building that Mr. Bridges had given them at that time, and so it was no surprise to him to see the pictures and diagrams of the very pleasing and efficient buildings that Mr. Bridges was now putting up. Such structures had set an entirely new conception for colliery surface layouts.

One thing which had struck him in connection with the design was the care which Mr. Bridges had exercised : he did not mean care in connection with the stability or the strength—one would assume that, of course—nor even with regard to the architectural features, but with the thought and trouble which had been given to the facilities for the installation, replacement and repair of plant. Such points made all the difference to the job of the local engineer who was going to have to live with those structures for a long time, and often such things had in the past been overlooked. With the old open type steel joist head frames and the detached colliery buildings there was not perhaps the necessity for that elaborate forethought, at least not to the same extent although probably everyone connected with collieries had seen more than one frame where far too much dismantling had been necessary in order to insert a new cage or even to change a guide rope, all because of lack of care in design. Everyone would be grateful to Mr. Bridges for setting out so very clearly the requirements which had to be borne in mind.

One minor point was that Mr. Bridges had not mentioned the circular headgear—those cylinders on the top of which was mounted the winding hoist and which were now becoming quite common practice in metalliferous mines, where possibly the ground was very much better for building purposes as far as the foundations were concerned. In the case of the circular headgears it was merely a continuation of the shaft walling, suitably strengthened as necessary. Although they were unusual in appearance they were not unpleasant, and sometimes a circular ore bin was built in at the side of the headgear for receiving the skip discharge, which gave additional stability.

Mr. T. N. W. Akroyd (Associate-Member) said since his particular interest was foundation engineering he hoped he would be excused if he dealt entirely with that part of Mr. Bridges' paper. Firstly, he was in complete agreement with all that the Author said in the paper, but he would stress the need for an investigation of the ground upon which the structure was to be founded, so that a sound and economic type of foundation could be designed. Not only was it necessary sometimes to put down trial loads, but it must indeed be a small industrial structure which did not warrant at least one bore hole being sunk. Whilst it was impossible to lay down rules to determine allowable bearing pressure it was certainly possible to carry out soil tests on samples taken during boring operations and from the results to calculate the allowable bearing pressure. Those calculations could then be confirmed by trial loadings where the ground immediately beneath the surface was the weakest. In fact unless the type of ground had been investigated it was probable that the results of a trial loading would be mis-interpreted.

Concerning piles, whilst he agreed that they were not the universal panacea for all foundation problems there were few soils in which some form of pile could not be used. Pile loading tests which were probably more common than the test loadings referred to by Mr. Bridges required the same strict caution in the interpretation of the results, for the stresses put into the ground by one pile were very different from those of a pile group.

As regards settlement, not only must it be tolerated for economic reasons but it had to be tolerated because it could not be prevented. In fact with some ground it was the allowable settlement which controlled the bearing pressure and the size and type of foundation rather than a consideration of the strength of the soil. It was usually considered that in the normal type of frame building the allowable differential settlement between adjacent columns might be of the order of $\frac{1}{2}$ in. to $\frac{3}{4}$ in. ; little data was available concerning industrial structures and he wondered whether Mr. Bridges had any information in this connection.

Although foundation engineering had for a long time been the Cinderella of structural engineering it was now possible, by means of a site investigation and soil tests, to determine the most suitable type of foundation ; in fact Mr. Akroyd said he would go further and say that unless an engineer carried out an investigation suitable for the type of structure to be erected then the structural engineer was not taking full advantage of the

experience gained during the last decade in that branch of engineering. After all, the structural engineer would not consider neglecting up-to-date information concerning steel or concrete and, therefore, why should he neglect information which could be obtained about the ground upon which he was to found his structure ?

Since the number of foundation failures which could be attributed to the difference between design assumption and what happened in practice was small, it must mean that the factor of safety was high and thus the foundation was not necessarily economically designed. Further information, obtainable only from site observation was required to ensure that there was the same control over the design of foundations as over the superstructure ; but the foundation engineer was dependent upon clients granting the necessary facilities for making such site observations.

Mr. G. P. Bridges, in replying to the discussion, said that as there was insufficient time available to reply to the points raised at the meeting he would reply fully to the discussion in due course. He very much appreciated the interest which had been aroused in the paper, and wished to thank all those who had taken part in the discussion.

It had been a great pleasure to undertake the task of writing the paper, and the interest shown had made it worthwhile.

Reply to Discussion

Mr. F. G. Glossop.—The headframe which Mr. Glossop referred to was built at Point of Ayr Colliery, and is shown in the photograph Fig. 7. The first interesting feature is that it was built before the sinking of the shaft proper was commenced, and by introducing a temporary framed strut on one side it was possible to use the frame for sinking purposes, and thus avoid the expense of building a temporary sinking frame. The second interesting point is that the ground at this shaft consists of fairly loose sand to a depth of about 90 ft. below the decking level, and as there was a risk of the sand becoming disturbed, it was decided to use Benoto Caisson Piles. There were six of these piles, each one metre diameter, and these were designed to carry a maximum load of 150 tons. This system of piling consists of sinking tubular steel caisons from which the ground is excavated and filled with reinforced concrete, at the same time withdrawing the steel tube.

The second illustration referred to by Mr. Glossop was the foundation for Compressor House built on piled foundations, driven through very bad ground which included strata of peat 7 ft. thick. There were four compressors, Bellis Morcam type, and each weighing approximately 54 tons. These compressors set up considerable vibration, and we were asked to design a foundation that would eliminate the risk of these vibrations being transmitted to the building. The arrangement of the foundations is shown in Fig. 8. It was decided to form a raft on the top of the piles covering the whole area of the building, and the foundations for the compressors were built on this raft, but separated from it by a membrane of cork insulation covering the whole area of each compressor foundation block. Similar insulation was provided between the supports of the concrete floor on the foundation blocks at the higher level. This detail proved quite satisfactory in preventing vibration transmission to the building.

The Author agrees with Mr. Glossop that reinforced concrete is preferable to steel as a structural material for headframes because of the reduced maintenance cost and the high resistance to vibration due to the continuity of the joints and connections, and also the generally low slenderness ratio of the members.

It is agreed that reinforced concrete does take longer in site erection, but very often this is balanced by saving in the time required for shop fabrication. The headframe illustrated above was built in readiness for the sinking operation in four months, and this illustrates what can be done when an effort is made.

Mr. G. Cuttle.—The diagrams produced by Mr. Cuttle illustrated very effectively the smaller space and height requirements of modern friction winders, and also showed how this affected the dimensions of the towers.

The smaller dimensions of the towers did, however, create additional problems in the design of the foundations, because the centres of the main columns are brought nearer to the shaft, with the result that the design of the fore-shaft lining and the columns must be considered in conjunction with the foundations.

Much discussion has already taken place on the merits of direct coupled as compared with geared driving units for friction winders, and these discussions will continue for a long time. In the case of geared drives in tower-mounted installations, deflections in the supporting beams has caused some concern, especially if the gears and motors are supported eccentrically on the beams. The arrangement of the structural members is, therefore, always important. Deflections cannot be entirely eliminated, but with careful design they can be kept within pre-determined safe limits.

Mr. Cuttle referred to the advantages of using temporary sinking headframes when it was necessary to use the freezing process in sinking operations. It is true that freezing does create additional problems because in freezing the ground expands and this must cause movements to any foundations in or near the freezing zone, conversely, when the ground is thawed shrinkage will take place and the foundations will tend to move toward their original positions. The easiest solution is partially to build the foundations before freezing takes place, and to complete them after the sinking is finished, so that any movement of the ground can be allowed for in fixing the foundation bolts or footings.

Mr. A. P. Mason.—Mr. Mason deserves our thanks for calling attention to the Koepe Winding installation at Murton Colliery, which was erected in 1923, and also to the installations which are practically completed at Bradford and Rothes Collieries. These illustrations emphasise very forcibly the Author's point in showing how slowly the application of friction winding developed before the war, and the amazing speed of development since the war, because the Bradford and Rothes installations were completely out of date before they were put to work, and it is very unlikely that any further single rope installations will ever be built. These remarks apply, of course, to the winding equipment, but as already pointed out the towers on a modern installation will be very much smaller.

Mr. H. C. Husband.—There is no legal or technical interpretation possible of what constitutes a temporary or semi-permanent structure, because the interpretation will change with each varying set of circumstances and with the opinions of each individual concerned. The point which needs emphasis is that Colliery equipment and Mine and Marketing methods change, and the buildings and structures which house this equipment

should be designed so that alterations and additions can be carried out with a minimum of disturbance.

The Author has no record of any Coal Preparation Plant Building being moved from one Colliery to another, but the process has occurred in connection with Screening Plants on many occasions. Quite a large number of Coal Preparation Plants have been built in recent years which have been designed with the express object of moving them to other sites when required.

In the past, cladding and housings for Coal Preparation Plants were designed to fit round the apparatus in order to give the desired protection from weather, to allow ample room for the operation of the plant, and for carrying out maintenance and repairs. In recent years the tendency is to design a building for the plant and to install apparatus after the building is completed. In the Author's opinion this is wrong because it results in a more expensive building, having a good deal of waste space and does not facilitate making changes as easily as would be expected. Very often, alterations and extensions have to be made to the buildings even before it is ready to start work and, therefore, the buildings and structural supports for the apparatus should be as flexible as possible.

One must agree with Mr. Husband in his remarks on the co-operation between the mechanical and structural engineers in preparing designs for towers—this was the only way to achieve satisfactory results. One must also agree with his remarks regarding concrete versus steel for winding structures. One cannot set down rules, for every case must be considered individually. For instance, it is very much easier to construct a steel framework when working round an existing tower or a sinking headframe, but if it is possible to use reinforced concrete then that is the best structural material for this particular job.

Mr. Geoffrey Palmer.—The Author was very pleased to meet again one of his old friends, and that he was sufficiently interested to be present at the meeting. Mr. Palmer stated that the N.C.B. should make up its mind whether it preferred temporary or permanent buildings. The question was not quite as simple as that statement inferred, because in most cases neither temporary nor permanent buildings were called for, and that is why the Author had introduced the term " semi-permanent " buildings. In the Author's opinion all engineers should use the most suitable structural material for the solution of the problem he is dealing with, and it is quite agreed that steel frames or reinforced concrete can be equally successful.

Another important point which he has stated is that modern friction winding practice followed very closely the lift design, only on a much larger scale, and it is interesting to note that several new winding installations had introduced balance weights as an alternative to balanced skips or cages. Attempts are also being made to use automatic and push-button control, so that winding can take place from any level at any time. The Author was not able to say why multi-rope winding had not been used earlier, but it is supposed that it is for the same reason that we did not use an aeroplane before the year 1900.

Mr. Liszka.—Dealing with the points raised :

1. The chief differences between drum and friction winders are, that in the case of the drum winder the rope is wound on the drum when raising the cage on one side, and off the drum for lowering the cage on the opposite side, but the rope is fixed to the drum. In the case of the friction winder the ropes are continuous, and are fixed only to the cages on skips which are supported directly on the friction wheel. This makes it possible for the winder to be mounted at the top of the tower, and there is no horizontal rope-pull to be taken.

2. The vibration is taken up in the structure by limiting the slenderness of the members, by taking full advantage of continuity at the connections, and using a lower design stress than would be used for a normal structure.

3. A structure with stiff joints and stiff members is obviously more suitable than a structure with stiff joints and slender members to take vibration in shock loading.

Professor J. C. Mitcheson.—The Author thanked Professor Mitcheson for his remarks on the design of some of the structures with regard to working facilities, maintenance, and repairs. These matters are of the utmost importance to the Colliery Engineer, and one does acquire a knack of providing these facilities after a long experience and, of course, many slips. Great care is taken today in the design of headframes and towers to make the changing of skips and cages or repairing equipment in the shaft, as easy as possible, and many Colliery Engineers take great pride in the speed with which they can change a skip or a cage.

The circular shaped headframes had been considered from time to time, but there were disadvantages in this type of design, and they were not always necessarily as economic in cost as would at first appear.

Mr. T. N. W. Akroyd.—The Author thanked Mr. Akroyd for his remarks on foundations which were, of course, universally agreed to.

Differential settlements of the magnitude of $\frac{1}{2}$ in. to $\frac{3}{4}$ in. were far greater than should be tolerated in any Colliery structure, although cases had occurred where settlements of this magnitude had taken place, and the corrections made by jacking the stanchions back into position as required. This can, of course, be more successfully done in the case of steel than in the case of concrete or brick structures.

Written Contribution

Mr. F. J. Samuely (Member): I have read with great interest Mr. George Bridges' paper on Colliery Structures, and I regret that I was not able to come to the meeting on April 28th.

There is one point which I should like to take up. Mr. Bridges mentioned that brickwork is the most satisfactory form of cladding for buildings of this type. My own experience is that $4\frac{1}{2}$ ins. brickwork can be used satisfactorily, together with a reinforced concrete framework, but that the combination of brickwork and uncased steel leaves much to be desired. Invariably certain points occur where the brickwork interferes with bracing, projecting flanges, etc., and where so much of it has to be cut away that very awkward connections occur. Also it is often very difficult to flash such points so that water is prevented from entering. From an aesthetic point of view these odd points are most revolting.

Many difficulties can be avoided by using 9 in. or even $13\frac{1}{2}$ in. brick cladding, but this is certainly a very expensive way of dealing with the problem.

With a rigid non-braced reinforced concrete structure with boot lintols, brickwork can be combined more

easily, but I have found that the use of large slabs of precast concrete, suspended from horizontal projecting flanges, overcome most of the problems, both in steelwork and reinforced concrete. Such precast slabs can either be made watertight by mastic or cement grout, or alternatively, they may overlap each other in the way sheet materials do. If there is sufficient repetition the cost of precast unit slabs is only a little more than that of 4½ in. brickwork.

Mr. BRIDGES writes : In reply to the contribution of Mr. Felix J. Samuely, 4½ in. thick brick cladding is most satisfactory in performance and cost, although it will be about 10s. per sq. yd. dearer than asbestos sheeting on steel framing. In modern construction it is usual to build the brick panelling completely outside the steel frame so that none of the structural members (except the window lintols) are exposed. This type of detail is to be preferred whether the framework is reinforced concrete or structural steel. Some ingenuity is required in designing the attachment of the panelling to the structural members and brick reinforcement may be required in the larger area panels.

Some difficulties may occur due to bracing, projecting flanges, etc., but if proper consideration is given to these details in the design of the framework they can be satisfactorily overcome without involving awkward flashing details or sacrificing aesthetic values. If it is essential for the building to be completely insulated against damp penetration 9 in. or 13½ in. brickwork may be necessary, but not otherwise.

Precast concrete panels are the best alternative to 4½ in. brickwork, but in most cases the cost is a little high. On account of the cost of expensive forms and heavy cranes for erection the cost of concrete panelling can be much higher than brickwork except where big areas are involved.

Errata

The following corrections have been noted in the paper as published in the April issue of the Journal :

Page 114. Column 2, Line 9—For " defector pulleys " *read* " deflector pulleys ".

Page 117. Paragraph 4, Line 9—For 6 *read* 8.
Paragraph 6, Line 12—For " from drum winder " *read* " for drum winder ".
Paragraph 6, Line 27—For " in place of ' buntings ' " *read* ' buntons '.

Page 118. Paragraph 3, Line 11—For " three-quarters " *read* " one and three quarters ".

Institution Notices and Proceedings

IMPORTANT NOTICE TO SUBSCRIBERS

Subscribers, Newsagents and Subscription Agencies at home and abroad are asked to note that as from the 1st January, 1955, copies of THE STRUCTURAL ENGINEER can only be obtained on application to the Secretary of the Institution of Structural Engineers, 11, Upper Belgrave Street, London, S.W.1.

Subscription Rate

The Annual Subscription, post free, to any part of the world is 44s. 6d.

Single Copies

The price of single copies is 3s. 6d. or 3s. 9d. if sent by post.

In order to fulfil all demands, subscribers and others are requested to state their requirements as soon as possible.

The usual rate of discount will apply.

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, November 24th, 1955

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when a Discussion on the Report of the Ad Hoc Committee on Structural Safety will be opened by the Chairman and members of the Ad Hoc Committee. (The Report which is the subject of the Discussion was published in the May issue of the Journal).

Thursday, December 15th, 1955

Ordinary General Meeting for the election of members, 5.55 p.m., followed at 6 p.m., by a Joint Meeting with the British Section of the Société des Ingénieurs Civils de France, when Monsieur R. Giguet will give a paper on " The Tignes Hydro-Electric Scheme."

Thursday, January 12th, 1956

Ordinary Meeting, 6 p.m., when Dr. R. K. Livesley, M.A., will give a paper on " The Application of an Electronic Digital Computer to some Problems of Structural Analysis."

Thursday, January 26th, 1956

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. F. M. Bowen, M.I.C.E., Assoc.I.Mech.E. (Member of Council) will give a paper on " Steelwork in Southampton Docks Transit Shed 102."

Members wishing to bring guests to the Ordinary and Joint Meetings referred to above are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS—JANUARY, 1956

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on January 10th and 11th, 1956 (Graduateship) and January 12th and 13th (Associate-Membership).

EXAMINATIONS—JULY, 1955

The examinations of the Institution were held in July, 1955, at the usual centres in Great Britain and overseas at :

Accra, Atbara (Sudan), Auckland, Beirut, Bombay, Bulawayo, Calcutta, Cape Town, Christchurch (N.Z.), Colombo, Delhi, Dunedin, Durban, East London, Gwelo, Hong Kong, Jerusalem, Johannesburg, Kampala, Karachi, Kuala Lumpur, Lagos, Lahore, Lucknow, Malta, Madras, M.E.A.F., Melbourne, Montreal, Mombasa, Nairobi, Ndola, Penang, Port Elizabeth, Rangoon, Salisbury (S. Rhodesia), Singapore, Sydney, Toronto, Welkom (South Africa), and Wellington.

One hundred and five candidates took the Graduateship Examination, (71 at home centres and 34 at overseas centres). Of these, 68 passed (48 at home centres

nd 20 at overseas centres). Five hundred and twelve andidates took the Associate-Membership Examination (402 at home centres and 110 at overseas centres). Of these, 158 passed (126 at home centres and 32 at verseas centres).

The names of the successful candidates are :

HOME CENTRES

Graduateship Examination

Allen, Derek.
Atkinson, John Stuart.
Bason, John Roland Thomas.
Bennett, Alvin Alphonso.
Bradnick, Robin James.
Brooke, Jack.
Brooks, Edward.
Brownlow, Jack.
Bull, Peter John.
Claxton, Brian Hugh.
Corfield, Cyril Thomas.
Crossley, Norman.
Evans, John.
Fawcett, Frank Birkinshaw.
Fearnley, James.
Floyd, Laurence William.
Francis, John Edwin.
Garrett, Maxwell John.
Goodchild, George.
Hill, Kenneth Wesley.
Howes, Roy.
Hutchinson, David Alan.
Jackson, Norman
Jones, Barry Alfred Champion.
Jones, Clifford Ivor.
Lambert, Michael Francis.
Lefever, Geoffrey James.
Levi, Albert.
Levick, Sidney Isaac.
Martin, Derek Kenneth.
Moore, John Neville.
Moore, Ronald Ernest.
Morgan, John Synford.
Oborne, David John.
Parrott, John Frank.
Platt, Kenneth John.
Punter, Terence Albert.
Ratcliffe, Keith Urwin.
Salmins, Artis Ilgonis Robert.
Smith, James Frank Hinton.
Speight, Peter A.
Stock, Stanley Gordon.
Swidzinski, Zbigniew Maria Michael.
Thomas, Martyn Robin Haden.
Tune, Donald Hector.
Ward, Denis Robert.
Wheaton, Roger Cecil.
Wilkinson, Frederick Alan.

Associate-Membership Examination

Ainsworth, Wilfred Edward.
Allan, Benon Jakup.
Allcott, Ernest James.
Bacon, Roger.
Bailey, Sidney.
Ballantyne, Desmond Trevor.
Banerjea, Amit Kumar.
Banerjee, Butto Krishna.
Barton, John Richard.
Beck, John Stanley.
Boswell, Carlton.
Brown, Leslie Arthur.
Burgess, John Holland.
Capps, Derrick Ernest.
Carling, Peter George.
Carter, Clifford.
Cartwright, John Desmond.
Case, John Kenneth.
Case, Lionel Francis.
Chaudhuri, Deb Kumar.
Clarke, Stanley.
Clifford, Paul Anthony.
Cohen, Maurice.
Craddock, Harry Norman.
Crease, David Plaistow.
Crossley, Norman.
Darlison, John Leonard.
Davies, Colin.
Davis, John.
Dean, Reginald.
Douglas, Roderick Emmanuel.
Downes, Henry George Edward.
Drummond, Brian Godfrey.
Dunthorne, Gareth John.
Durkin, Kenneth Ronald.
Ehsanullah, Ehsan.
Ellis, Tony James.
Evans, Glyn.
Evans, Ronald John.
Farmer, Arthur George.
Farnborough, Robert Furnberg.
Feakes, Gerald William Ernest.
Feit, Michael John.
Fishwick, Adrian Lee.
Folland, Alfred John.
Fowke, Bertram William.
Foxwell, John.
Gardner, Roger Whiteside.
Gee, Jeffrey Ernest Malcolm.
Ghosh, Asish Kumar.
Ghosh, Madan Mohan.
Gibson, John Hartt.
Gliniecki, Zbigniew Marian.
Goddard, Philip Dennis.
Goodier, Roger Ewart.
Graney, Dennis Francis.
Green, Ernest.
Hadjicosta, George Costa.
Hardiman, Noel James.
Henderson, John Graham.
Henderson, James Keir.
Higgs, Thomas Fallon.
Higham, Roland.
Hingorani, Narain Vasiomal.
Hixson, Robert Ivor.
Hoare, John Hubert.
Hobin, Richard.
Hogan, Gerald.
Horwell, David Gordon.
Hotz, Peter Martin.
Husain, Mahmood.
Huscroft, Leslie.
Idowu, Charles Olatunji.
Jolley, Dennis.
Kirk, Sidney Edward.
Kwaku, Samuel Francis.
Lake, John Brian Francis.
Langdon, John.
Lau Yat Sun.
Lawson, Donald.
Lethbridge, John Anthony.

Lewis, Wyndham Stuart.
Lord, Percival George.
McKelvey, William Leslie.
MacKenzie, Ian Gilroy.
McNally, Eneas.
Martin, Geoffrey Philip.
Mercer, George.
Minney, Arthur Joseph.
Mitchell, John Philip.
Mukherjee, Kunjalal.
Munro, John.
Murphy, Kevin Malcolm.
Neave, Harry.
Nuttall, Peter.
Parton, Girvan Meir.
Patel, Indubhai Varadhbhai.
Phalp, Brian Trevor.
Pinkney, John Dennis.
Prior, Geoffrey.
Revesz, Stephen.
Robson, John Rowland.
Russoff, Barry.
Shuttleworth, Bernard Thomas.
Simpson, John Edward Henry.
Smethurst, Gordon James.
Spedding, Alan Henry.
Symes, Robert Francis.
Szelepet, Jan.
Thair, Arthur Ernest.
Thompson, Donald.
Thomson, Andrew Gordon.
Tonge, Brian Yardley.
Traverse, John Derek.
Turner, Colin Francis.
Vijiaratnam, Arumugam.
Watson, John Rex.
Wehrle, John Emile Charles.
Weller, Alan David.
Wells, William John.
White, Allen Robert.
Wilkins, John Richard.
Wilkinson, Herbert.
Williams, Geoffrey Theo.
Wojakowski, Andrew Walter Stanley.
Yap Ee Siew.

OVERSEAS CENTRES.

Graduateship Examination

Allon, Isac.
Coleman, George Martin.
Forrest, Esli James.
Gandhi, Kaikobad Manecksha.
Gandhi, Vashu Jiaram.
Gentry, Leslie John.
Goodey, Donald Alfred.
Herman, Terence Francis.
Jones, Brian Vernon.
Lau Yau Chun.
Mehta, Navinchandra Raghunath.
Mustapha Bin Hussain.
Nicholas, Wilfred Arasaratnam.
O'Grady, Carl Robert.
Roberts, Joseph Frank Robert.
Singh, Matharu Mohan.
Slack, George Edwin.
Stirling, Ronald Robert Sinclair.
Thurston, Clifford Charles.
Tsang Yin Sang.

Associate-Membership Examination

Adams, Hugh Morley.
Archibald, Allan Edwin.
Baikoff, Eugene Marc Alexander.
Bromley, Victor John.
Cheng Chung Chow.
Crawford, James Henry.
Du Toit, Pieter Schalk.
Farley, Michael George Gilbert.
Fraser, Angus Stuart.
Gibbs, Douglas Rutherford.
Grahame, Russell Godfrey.
Herring, Bernard Carroll.
James, Bryan Wilson.
Jha, Krishna Mohan.
Kotnis, Vishvanath Ramchandra.
Kung Shien-Cho.
Lahiri, Ramapada.
McIlroy, William Robert.
Moses, Charles.
Naik, Dinkar Baburao.
Nightingale, Alan Francis.
Phanse, Arun Prabhakar.
Pokorny, Milos Karel.
Robertson, Alexander Stuart.
Sarkar, Suniti.
Shunn, William Gordon.
Sneath, Norman.
Thomason, Dennis William.
Tonner, Eric William.
Tsang Foh Kwei.
Tucker, Kenneth Charles.
Wilson, Daniel Russell.

RULES OF CONDUCT

The Council having received a complaint against a member alleging a breach of Clause 4(g) of the Rules of Conduct, a specially convened meeting was held in order that the member might be given the opportunity of answering the charge made against him. The member accompanied by his legal advisers attended the meeting and witnesses were interrogated.

The Council found the charge against the member proved and unanimously resolved that he be reprimanded and informed that if the offence were to be repeated, they would have to take a much more serious view of his position, since apart from any stigma which might be attached to the member concerned, any wrongful action would affect the reputation of all members of the Institution.

LIBRARY FACILITIES AVAILABLE TO INSTITUTION MEMBERS

The Institution has a small but specialist library of some 3,500 volumes, dealing mainly with structural engineering but including some applied subjects, and containing some rare books.

Most of the more important textbooks on structural engineering are available and also the proceedings or transactions of a number of engineering institutions and societies. In addition, large numbers of pamphlets and papers are filed in the library, and nearly one hundred technical journals are received regularly and are available to members. Almost complete sets of the *Engineer and Engineering*, dating back to the first issues, are in the reference library, and copies of other important journals covering a number of years are also filed.

Members of the Institution may borrow any book or pamphlet which is in general circulation, and also copies of transactions and periodicals, either by making personal application or by post.

By an arrangement with the Science Museum Library, books can be obtained on loan for the use of members, and the Institution is also in touch with the libraries of other Government Departments and a number of other important technical libraries.

In addition to the index of books, the librarian keeps an up-to-date card index of papers on structural engineering and allied subjects and of articles which have appeared in the technical press, and bibliographies on particular subjects are compiled from these indexes on application, and assistance given in connection with technical inquiries.

The Institution also maintains a card-index of abstracts giving summaries of papers appearing on structural engineering subjects throughout the world.

MACLACHLAN LECTURE COMPETITION, 1956

The closing date for the receipt of entries for the next Maclachlan Lecture Competition is Friday, March 30th, 1956. The general conditions of the competition are as follows :

1. The Institution of Structural Engineers shall institute a written lecture to be known as the Maclachlan Lecture and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Engineering as long as in every second year the subject shall be confined to steel structures.

3. Entrance into the competition for the Lecture shall be confined to Graduates and Associate Members of the Institution who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning Lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award, or for any other reason, the Institution shall transfer the above sums to the Research Fund of the Institution.

Particulars of the Competition for 1956

1. The Maclachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1956.

2. The subject of the Lecture shall be on any aspect of structural engineering.

3. The work should be submitted as the script of a lecture which the author, if successful in the competition, will deliver before an audience in the course of about one hour. The development of mathematical formulæ and detailed calculations should be avoided as far as possible in the text ; if they are essential they should be embodied in appendices. Photographs, drawings, graphs, etc., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.

4. Six copies of each Lecture should be submitted and should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the Institution is Friday, March 30th, 1956.

LONDON GRADUATES' AND STUDENTS' SECTION

The next meeting of the Section will be held at 11, Upper Belgrave Street, London, S.W.1., on Wednesday, November 9th, 1955, at 6.30 p.m., when Mr. G. I. Goulden, I.A., A.R.I.B.A., Deputy Director and Chief Technical Officer to the Building Centre, will talk on " Engineering in Architecture."

A meeting will be held on Monday, December 19th, when Mr. G. A. Wilson, M.Eng., M.I.C.E., M.I.Mech.E., Chief Engineer, Port of London Authority, will talk on " Engineering Problems in the Docks."

Hon. Secretary : D. E. Capelin, 40, Thornton Crescent, Old Coulsdon, Surrey.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :

Wednesday, November 9th, 1955

Joint Meeting with the Liverpool Metallurgical Society at Picton Hall. Dr. A. A. Wells on " Steel and Welding."

Monday, November 14th, 1955

At the College of Technology, Manchester. Mr. G. Bernard Godfrey, A.M.I.C.E., A.M.I.Struct.E., on " Joints in Steel Rigid Frameworks."

Wednesday, November 30th, 1955

At the Liverpool Engineering Society, Dale Street, Liverpool. Mr. J. Singleton-Green, M.Sc., M.I.C.E., A.M.I.Mech.E. (Member of Council), on " Concrete as an Engineering Material."

Thursday, December 8th, 1955

At the College of Technology, Manchester. Mr. F. M. Little, B.Sc.(Tech.), A.M.I.C.E., A.M.I.Struct.E., on " The Influence of the Motor Vehicle on Urban Buildings."

Tuesday, January 10th, 1956

Joint Meeting with the Liverpool Branch of the Institute of Welding. Professor W. J. Kearton, D.Eng., M.I.N.A., M.I.Mech.E., on " Photoelasticity." At the University of Liverpool.

Thursday, January 12th, 1956

At the College of Technology, Manchester. Mr. Felix J. Samuely, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E., on " Space Frames and Skin Structures."

Wednesday, January 18th, 1956

At the Liverpool Engineering Society. Mr. A. Bolton, M.Sc.,(Tech.) (Graduate), on " The Calculations of Critical Loads for Rigid Frames."

Unless otherwise stated, the above meetings will commence at 6.30 p.m., preceded by tea at 5.45 p.m.

Joint Hon. Secretaries: J. L. Robinson, A.M.I.Struct.E., 314, Northenden Road, Sale, Manchester; M. D. Woods, 58, Spring Gardens, Salford, 6.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :

Tuesday, November 1st, 1955

Joint Meeting with the Reinforced Concrete Association, Midland Counties Branch, at the Midland Institute, Birmingham. Mr. E. Shepley, B.Sc., A.M.I.C.E., on " Design of a Building."

Thursday, November 3rd, 1955

At the Becket Sale Rooms, Derby, 7 p.m. Mr. W. C. Clenshaw, D.I.C., B.Sc.(Eng.), A.M.I.C.E., on " Derby Station—Reconstruction of Platform Roofs."

Friday, November 25th, 1955

Joint Meeting with the Reinforced Concrete Association, Midland Counties Branch. Mr. J. E. C. Farebrother, M.I.Struct.E., on " Prestressed and other Reinforced Concrete Work at Ansells Brewery."

Saturday, November 26th, 1955

Works visit to Ansells Brewery to inspect plain and prestressed reinforced concrete work described in the paper by Mr. Farebrother. Assemble at 10.30 a.m.

Friday, January 27th, 1956

Mr. P. M. Worthington, B.Sc., on " The Design of Crane Gantries and the Action of Cranes on the Tracks."

Unless otherwise stated, meetings will be held at the James Watt Memorial Institute, Birmingham, and will commence at 6 p.m.

Hon. Secretary: L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES GRADUATES' AND STUDENTS' SECTION

Hon. Secretary: J. E. Jeffries, 18, Radnor Road, Handsworth, Birmingham, 23.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :

Tuesday, November 1st, 1955

At Middlesbrough. Mr. H. Saunders, A.I.Struct.E., on " Castellated Construction."

Wednesday, November 2nd, 1955

The above meeting will be repeated at Newcastle.

Tuesday, December 6th, 1955

Meeting at Middlesbrough—details to be arranged.

Wednesday, December 7th, 1955

Meeting at Newcastle—details to be arranged.

Wednesday, January 4th, 1956

Joint Meeting with Northern Architectural Association. Mr. D. R. R. Dick, B.Sc., M.I.C.E., on " The Civil Engineer and Britain's Atomic Factories."

Wednesday, January 11th, 1956

Joint Meeting with the Institution of Civil Engineers. Mr. D. R. R. Dick, on " The Civil Engineer and Britain's Atomic Factories."

Thursday, January 31st, 1956

At Middlesbrough. Mr. F. W. Slatter, M.I.Struct.E., and Mr. A. Brown, A.M.I.Mech.E., on " Foundations, Underpinning and Structural Problems at the ' Daily News ' Building, London."

Unless otherwise stated, Middlesbrough meetings will be held at the Cleveland Scientific and Technical Institution, Corporation Street, Middlesbrough, and Newcastle meetings at the Neville Hall, Westgate Road, Newcastle-upon-Tyne. Meetings will commence at 6.30 p.m., preceded by buffet tea at 6 p.m.

Hon. Secretary: O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :

Tuesday, December 6th, 1955

At the College of Technology, Belfast, 6.45 p.m., preceded by tea at the Overseas League premises, Wellington Place, Belfast. Mr. C. Munro, A.R.I.B.A., on " The Architect and the Structural Engineer."

Tuesday, January 3rd, 1956

Lecture, to be arranged.

Hon. Secretary: A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., " Barbizon," 2, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :

Tuesday, November 8th, 1955

Joint Meeting with the East of Scotland Branch of the Institute of Welding, at the Adelphi Hotel, Coburn Street, Edinburgh, 6.15 p.m. Mr. James McLean, B.Sc., on " Welded Plate Structures."

Tuesday, December 6th, 1955

At the Institution of Engineers and Shipbuilders, Elmbank Crescent, Glasgow, 7 p.m. Mr. Donovan H. Lee, B.Sc.(Eng.), M.I.C.E., M.I.Mech.E., M.I.Struct.E., M.Am.Soc.C.E., on " The Advantage and Disadvantages of Structural Steelwork, Reinforced Concrete and Prestressed Concrete."

Tuesday, January 17th, 1956

At the Institution of Engineers and Shipbuilders, Glasgow, 7 p.m. Mr. D. V. Pike, M.I.Struct.E., A.M.I.C.E., on " Aluminium Alloy Structures."

Hon. Secretary: W. Basil Scott, M.I.Struct.E., 19, Waterloo Street, Glasgow, C.2.

SOUTH-WESTERN COUNTIES BRANCH

The following meetings have been arranged :

Saturday, November 5th, 1955

At the Duke of Cornwall Hotel, Plymouth, 6.30 p.m. Chairman's Address by Lt.-Colonel R. Hazzledine, O.B.E., T.D., M.I.Struct.E. The President and the Secretary of the Institution will attend the meeting which will be followed by an informal Dinner.

Friday, December 16th, 1955

Meeting at Exeter, when a paper on Timber Construction will be given.

Friday, January 20th, 1956

At the Duke of Cornwall Hotel, Plymouth, 7 p.m. A paper on " The Development of Plastic Design " by Professor J. F. Baker, O.B.E., M.A., D.Sc., M.I.C.E., M.I.Struct.E., will be read by Mr. M. R. Horne, M.A., Ph.D., A.M.I.C.E., on behalf of the Author.

Joint Hon. Secretaries : E. W. Howells, M.I.Struct.E., 10/12, Market Street, Torquay, Devon. C. J. Woodrow, J.P., " Elston," Hartley Park Villas, Tavistock Road, Plymouth.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :

Tuesday, November 1st, 1955

At the South Wales Institute of Engineers, Park Place, Cardiff, 6.30 p.m. The Chairman, Mr. A. V. Hooker, M.I.Struct.E., A.M.I.C.E., will repeat his Address : " Recent Trends in the Design of Industrial Buildings."

Saturday, November 12th, 1955

At the County Buildings, Colwyn Bay, 6 p.m. The Chairman's Address will be repeated.

Thursday, December 15th, 1955

At the South Wales Institute of Engineers, Cardiff, 6.30 p.m. Joint Meeting with the Institution of Civil Engineers. Mr. F. Walby, M.I.C.E., and Mr. H. C. Adams, M.I.C.E., on " Prestressed Concrete as applied to Building Frames."

Wednesday, January 18th, 1956

At the Mackworth Hotel, Swansea, 6.30 p.m. Major A. F. Allen, M.I.C.E., on " Description of Neath By-pass Constructional Work."

Hon. Secretary : K. J. Stewart, A.M.I.C.E., A.M.I.Struct.E., 15, Glanmor Road, Swansea, Glam.

WESTERN COUNTIES BRANCH

The following meetings have been arranged :

Thursday, November 17th, 1955

Combined Meeting with the Institution of Civil Engineers. Mr. C. Parry on " Shuttering."

Friday, December 2nd, 1955

Papers by Branch members : Mr. J. A. Young, A.M.I.C.E., A.M.I.Struct.E., on " Joints in Water Retaining Structures with particular reference to Reservoirs " and Mr. N. T. A. Beavan, A.M.I.Struct.E., on " Co-operation between the Consultant and the Steelwork Engineer."

Friday, December 9th, 1955

Combined Dance, Royal Hotel, Bristol.

Friday, January 6th, 1956

Mr. H. W. Moore, M.I.Struct.E., will give a paper on a subject to be announced later.

Unless otherwise stated, meetings will be held in the Small Lecture Theatre in the new Engineering School, University of Bristol, at 6 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary : E. Hughes, M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The following meetings have been arranged :

Wednesday, November 16th, 1955

Joint Meeting with the Yorkshire Association of the Institution of Civil Engineers, at the University, Leeds, 6.30 p.m. Mr. J. A. Banks, O.B.E., M.I.C.E., on " The Allt-na-Lairige Prestressed Concrete Dam."

Wednesday, December 21st, 1955

Mr. J. Robertshaw, B.Sc., A.M.I.C.E., on " Geophysical Methods of Exploration and their application to Engineering Problems."

Wednesday, January 11th, 1956

Joint Meeting with the Yorkshire Association of the Institution of Civil Engineers at the Blue Bell Hotel, Scunthorpe, 6.30 p.m. Mr. G. M. Boyd, A.M.I.Struct.E., on " Brittle Fracture Problems in Steel Construction."

Wednesday, January 18th, 1956

Details to be arranged.

Unless otherwise stated, meetings will be held at the Great Northern Hotel, Leeds, at 6.30 p.m.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box No. 3306, Johannesburg, South Africa.

During weekdays Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. Phone 34-1111, Ext. 257.

Natal Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o the Reinforcing Steel Co. Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary: T. Stubbs, M.I.Struct.E., African Guarantee Building, 8 St. George's Street, Cape Town.

ADDITIONS TO THE LIBRARY

A.S.T.M. Proceedings, Vol. 54, 1954. Philadelphia, 1955, 1412 pp.

BRAITHWAITE & Co. STRUCTURAL LTD. *Pressed Steel Tanks*. London, 1955, 76 pp.

BRITISH IRON & STEEL RESEARCH ASSOCIATION. *The Prevention of Corrosion*. London, 1955, 20 pp.

BRITISH WELDING RESEARCH ASSOCIATION. *The Plastic Moduli of British Standard Rolled Steel Joists*. London, 1955, 10 pp.

CEMENT AND CONCRETE ASSOCIATION. *Proceedings of a Symposium on Mix Design and Quality Control of Concrete*, 1954, 548 pp. London, 1955.

Concrete Roads : (1) Earthworks, Sub-Grades and Bases ; (2) Slab Design ; (3) Design of Mixes ; (4) Construction Methods. pp. 15, 13, 16 and 31 resp. London, 1955.

Tech. Rept. No. TRA/186 The Ultimate Strength of Two-Span Continuous Prestressed Concrete Beams as affected by Tendon Transformation and Un-Tensioned Steel, by P. B. MORICE and H. E. LEWIS. London, 1955.

Tech. Rept. No. TRA/193 Local Effects of Concentrated Loads on Bridge Deck Slab Panels, by P. B. MORICE, London, 1955.

Library Translation No. 54. Provisional Paper on Joints in Concrete Roads. London, 1955.

CONCRETE ASSOCIATION OF INDIA. *Concrete Structures in India*, Bombay, 1955, 74 pp.

CUSSENS, A. R. *The compaction of fresh concrete by vibration*. Thesis for University of London, 1955. Presented by Professor H. J. Collins.

D.S.I.R. REPORT OF THE BUILDING RESEARCH BOARD, 1954, London, 1955.

Forest Products Research Special Rept. No. 9. The Strength of Timber Struts. London, 1955.

Forest Products Research Special Rept. No. 10. Laminated Beams from two species of Timber.

Theory of Design, London, 1955.
National Physical Laboratory Notes on Applied Science No. 11. Wind Effects on Bridges and other Flexible Structures, London, 1955.

FISHER AND LUDLOW. *Fisholow Flooring Systems.* Birmingham, 1955, 83 pp.

FRASER, J. M. *The Work of the Singapore Improvement Trust,* 1953, Singapore, 1953.

HAN, Maung Mya. *Correlation of California Bearing Ratio and Triaxial Compression Results for Selected Soils.* Thesis for University of London, 1955. Presented by Professor H. J. Collins.

MCNAMEE, J. *Analysis of Symmetric Cylindrical Shells : Its Application to Civil Engineering Design.* London, 1955.

NASR, Mahmoud, *Berechnungsweise von Spannbetonkonstruktionen in den verschiedenen Landen* (Methods of Calculation for Prestressed Concrete in Various Countries). Presented by the Author.

PRIEST, H. M. and GILLIGAN, J. A. *Design Manual for High Strength Steels.* U.S.A.: United Steel Corporation, 1954, 174 pp.

Royal Institute of British Architects. Report of a Symposium on High Flats. London, 1955, 47 pp.

STUSSI, F. *Die Theorie der Dauerfestigkeit und die Versuche von August Wohler.* Zurich, 1955, 48 pp. Presented by the Author.

TIMOSHENKO, S. P. *History of Strength of Materials :* With a brief account of the History of Theory of Elasticity and Theory of Structures. New York and London, 1953. Presented by Mr. J. M. G. Forsyth.

TIZARD, Sir H. *A Scientist in and out of the Civil Service.* Haldane Memorial Lecture. London, 1955.

United Steel Structural Co. Ltd. Illustrated Booklet, Scunthorpe and London, 1955.

University of Illinois Bulletins. No. 52 Reprint. *Research on Highway Bridge Floors,* 1936-54, by N. M. NEWMARK and C. P. SIESS. No. 428, *Strength in Shear of Reinforced Concrete Beams,* by A. LAUPA, C. P. SIESS and N. M. NEWMARK. No. 429, *Observed and Computed Settlements of Structures in Chicago,* by R. B. PECK and M. E. UYANIK. No. 430, *Stresses Around Mine Openings in Some Simple Geologic Structures,* by R. D. CAUDLE and G. B. CLARK. Circular No. 66, *The Axial Variation of the Magnetic Field in Solenoids of Finite Thickness,* by D. E. MAPOTHER and J. N. SNYDER.

Book Reviews

(*continued from page 349*)

Constructional Steelwork Simply Explained (3rd Edition), by Oscar Faber. (London : Oxford University Press, 1954.) $8\frac{3}{4}$ in. × $5\frac{3}{4}$ in., 138 pp. Price 12s. 6d.

The third edition of this standard elementary textbook is brought up-to-date to comply with the latest British Standard Specification. Thus the permissible stresses have been revised, some methods of calculation have been changed, the examples have been revised and new curves and figures added. A new appendix, giving a short summary of current permissible stresses has been included.

Elementary Engineering Mechanics, by J. B. Thirlwell. (London : Macdonald, 1955). 126 plus × pp., 8 in. × 5 in., 10/6d.

This book, as the title infers, is an elementary one and will be found extremely useful for beginners.

The author is to be congratulated on his clarity of expression of all the principles involved. He explains very thoroughly, such items as Velocity, Acceleration, Forces, Moments of forces, Work and Power, Energy, Momentum and Simple machines.

The book is very well illustrated and the line diagrams are reproduced in bold, clear outlines and will be found easy to follow by any beginner. It should appeal especially to those students desirous of taking up instrument-making, and horology, etc., since special attention is directed to these subjects involving mechanics for small classes of work, although the basic principles apply equally well to general engineering and that of heavier duty.

A list of terms used in engineering (and abbreviations) is also incorporated, by means of which the student should soon become conversant and various exercises, together with answers, are set at the end of the book. It shows clearly how the author thoroughly appreciates the difficulties experienced by some beginners in grasping the basic principles since he explains all the points in detail.

A. E. P.

"Concise Practical Surveying," by William Curtin, A.M.I.Struct.E., and Robert F. Lane, A.R.I.C.S. (English Universities Press Limited, 1955). 169 pp., $8\frac{1}{2}$ in. × $5\frac{1}{2}$ in. 15/- net.

In all branches of the professional side of the Building and Civil Engineering Industries more than a slight knowledge of Land Surveying is requisite, and not only the student and the more advanced man, but all those engaged on the technical side will find this little book useful and a handy reference. Its elementary opening chapters on chaining and levelling are within the grasp of even those who have only an ordinary knowledge of Euclid and trigonometry; and whilst leading the student on to field work the book explains lucidly the measurement of angles, transverse surveying, curve ranging, areas, volumes and plotting, as well as the use and adjustment of the instruments, and, if needed, rectification of errors. Of course, the fundamental is accurate field work but the many sketches and numerous examples of problems and their solutions if faithfully followed will make the labour easy. The authors have dealt with the subject in a concise and masterly way, and at the same time made the book interesting.

The book is one of the General Technical Series and can be used by those studying technical subjects for the General Certificate of Education.

H. H. B. S.

Presidential Address—Session 1955-1956*

Twenty-one Years' Progress as a Chartered Institution

By Stanley Vaughan, B.Sc., M.I.C.E., A.C.G.I.

1. Preliminary

ON 4th May, 1934, a Royal Charter of Incorporation was conferred upon the Institution of Structural Engineers by his late Majesty, King George V.

On 4th May, 1955, therefore, the Institution completed 21 years as a Chartered Institution. It was felt that this 'coming of age' anniversary should not be allowed to pass without recognition in the Presidential Address, and this led inevitably to the choice of subject and title for the present Address.

I should like to make it clear at the outset that although I shall necessarily refer to some of the outstanding clauses of the Charter, this Address is not intended to be an examination of the Charter itself, nor an exposition of its full implications to us as Structural Engineers, as this has been so admirably dealt with already by Dr. Fisher Cassie in his Chairman's Address, entitled "Charter Counsel," presented at the Inaugural Meetings of the Northern Counties Branch, first at Newcastle on 8th October, 1947, and then at Middlesbrough on the following day. This outstanding and inspiring Address was published in the February, 1948, issue of "The Structural Engineer" (Vol. XXVI, No. 2) and could, with advantage and interest, be read (or re-read) by all members of the Institution.

In the present Address I am proposing to invite you all to step aside with me and take a peep at ourselves from different angles, and in this way build up a mental picture showing how we have developed and progressed, during the last 21 years, along the direction so clearly set out for us in the Charter.

There are, however, two matters on which I would crave your indulgence in advance.

In the first place, time limitations clearly make it necessary for me to select from an immense volume of information available on the subject of our progress and development during the Charter period, and I may, therefore, be obliged to exclude many items which individual members might have preferred to have been included.

In the second place, many members, particularly the more senior members, will necessarily be conversant with many of the facts presented, but it is hoped that the method of presentation may help to throw some fresh light on these facts. Moreover, it is proposed to collect these facts together within one frame and to arrange them in an order which it is hoped will present the intended 'picture.'

Before proceeding with this review of our progress during the Charter years however, I propose to add a few words about our pre-Charter years. It was due to the devoted efforts (often in the face of great indifference and discouragement) of our early Presidents, Councils, Committees and members generally and also of our permanent staff, that the Institution grew from its inception to a position of sufficient prestige and importance to enable it to apply for and be granted a Royal Charter in 1934.

I should therefore like to record on your behalf our great thanks to those early stalwarts (some of whom are still with us) who gave so much of their time, wisdom and energy to building up this Institution on so sure and firm a foundation, and who never lost faith in the great future which awaited it.

2. The Charter and Bye-Laws

Perhaps the most vital Clause of the Charter is Clause 4 which defines the 'objects and purposes' for which the Institution was constituted.

Five separate 'purposes' are set out, and of these the first is so clear and so comprehensive that it embraces by implication the more detailed and specific four remaining 'purposes,' and I therefore offer no apology for quoting this first purpose in full. It reads :

"To promote the general advancement of the science and art of structural engineering in any or all of its branches and to facilitate the exchange of information and ideas relating to structural engineering amongst the members of the Institution and otherwise."

This clear and comprehensive statement of 'purpose' is the guiding beacon for all the activities of the Institution.

The only other Clause of the Charter to which I wish to draw your particular attention is Clause 18, which required the Institution to make Bye-Laws defining the rights and privileges of members and to submit them within a prescribed period to the Privy Council for their approval. Such Bye-Laws were duly drafted and submitted to the Privy Council, and approval obtained. Provision was also made in the Charter for future revisions to be made to these first Bye-Laws, if and when desired by the Institution, to meet changing conditions and requirements, subject always to approval by the Privy Council; and advantage has been taken of this provision on four occasions, the latest revisions having been made in 1954.

The Charter and the Bye-Laws are closely interdependant documents and should be studied jointly. Together they describe clearly and precisely the "Limits of Deviation" within which it was intended the Institution of Structural Engineers should function and develop, including full particulars of conditions of admission to each of the classes of membership and the duties, rights and privileges

* *Given before a General Meeting of the Institution of Structural Engineers at* 11, *Upper Belgrave Street, London, S.W.*1 *on the* 6*th October*, 1955.

attaching to each of these classes. But clear and precise as these documents are, and in fact largely because of this very precision, the closest study of them will not give more than a knowledge of the bare framework on which the Institution's activities are built. It is my intention to try and clothe this well designed but skeleton framework so as to give a more complete picture of the human and active body known as the Institution of Structural Engineers which has developed around this framework.

3. Geography

Let us first of all consider the geographical distribution of the Institution's activities.

In the first place this London Headquarters building serves not only as the meeting place for the Council and Committees and the offices where the Secretary and his staff function, but it also provides the Meeting Hall where General Meetings of the Institution are held, and where papers are presented to the Institution. The Institution library is also housed in the London Headquarters building.

In addition to serving the activities of the Institution as a whole, the London Headquarters acts as a sort of branch headquarters to London members.

At the time when the Charter was granted there were, in addition to London Headquarters, seven separate Branches operating (and I name these in alphabetical order for obvious reasons) :

Lancashire and Cheshire Branch.
Midland Counties Branch.
Scottish Branch.
South Wales Branch (later changed to Wales and Monmouthshire Branch).
South-Western Counties Branch.
Western Counties Branch.
Yorkshire Branch.

Since the Charter date three further branches have been formed :

1937—Transvaal Branch (later enlarged and renamed the Union of South Africa Branch).
1947—Northern Counties Branch.
1948—Northern Ireland Branch.

The geographical extent of each of the branches in the United Kingdom and Northern Ireland is shown in Fig. 1. This also shows the locations of the Headquarters centres for these branches. It will be seen that in two cases a Branch has two 'centres,' namely Newcastle and Middlesbrough in the case of the Northern Counties Branch, and Cardiff and Swansea in the case of the Wales and Monmouthshire Branch. In addition, this latter branch has a sub-section covering North Wales, meetings of which are held at Colwyn Bay.

I offer my apologies to the Union of South Africa Branch that it is not possible to demonstrate the extent of their territory in Fig. 1—purely for reasons of scale, but I have taken care to include this branch in the table given on the figure which shows the present numerical strength of each Branch, and I would like to take this opportunity of letting this Overseas Branch know how proud we are to have such a large and vigorous Branch of the Institution in another Continent.

In addition to the Branches described, the Institution has this year received a request from members in Malay to be allowed to form a Branch with Headquarters in Singapore. Unfortunately the number of full Members available in that area was not sufficient to permit the formation of a Branch under the Bye-Laws. The Institution was, however, able to recommend in strict accordance with the Bye-Laws the formation of a 'Malayan Section' of the Institution with a prospect of transfer to 'Branch' status at some future date when sufficient full Members are available to make this permissible, and it is hoped that this recommendation may bear fruit.

4. Organisation and Administration

In accordance with Clause 10 of the Charter, the business of the Institution is managed by the Council, subject to the provisions of the Charter and the Bye-Laws.

The Council is a fully elected body (by annual secret ballot amongst all corporate members, i.e. full Members and Associate Members).

The constitution of the Council is laid down in Bye-Law 70 and a study of this will show that great care has been taken to ensure that all sections and all interests of the Institution are fairly represented.

Provision is made for the equivalent of the "elders of the kirk" by the inclusion of the six most recent Past-Presidents, so making available in Council the accumulated experience of six members, each of whom must have spent many years in helping to deal with the affairs of the Institution.

To assist in dealing with the Institution's business, the Council has appointed Standing Committees, each of which considers and reports to the Council on matters relating to the particular activities of the Institution which lie within its own terms of reference.

Of these Standing Committees, the five which meet most regularly (normally once a month) to deal with the day to day business which arises, are the following :

Finance and General Purposes Committee,
Membership Committee,
Literature Committee,
Science and Research Committee,
Education and Examinations Committee.

In addition there are the following Standing Committees which meet as and when required :

Legislation Committee,
Bye-Laws Committee,
Nominations Committee,
Practice Committee,
Special Committee on Awards.

Every effort is made to ensure that all activities of the Institution and every happening in the structural engineering world which might affect either the Institution as a whole or any individual member of it comes before one or other of the Standing Committees. The matter is debated in Committee and the findings or recommendations of the Committee are reported to the Council. Here the matter is again laid open for discussion to fresh minds, with perhaps a wider range of outlook, and the Committee's recommendations are then either confirmed, modified, or referred back to the Committee for further consideration.

When a question comes before any of the Standing Committees which requires considerable work and clos

BRANCHES	CENTRES	TOTAL NUMBERS IN 1954
LANCASHIRE & CHESHIRE	MANCHESTER	752
MIDLAND COUNTIES	BIRMINGHAM	624
NORTHERN COUNTIES	NEWCASTLE MIDDLESBROUGH	262
NORTHERN IRELAND	BELFAST	38
SCOTTISH	GLASGOW	308
SOUTH WESTERN COUNTIES	PLYMOUTH	79
UNION OF SOUTH AFRICA	JOHANNESBURG	401
WALES & MONMOUTHSHIRE	CARDIFF SWANSEA	167
WESTERN COUNTIES	BRISTOL	136
YORKSHIRE	LEEDS	324

Fig. 1

study before a fair opinion can be formed, the Committee appoints a small Sub-Committee from its members (with, if necessary, co-opted members) to make the necessary close study and to report back to the main Committee.

When a subject arises of major importance which may deeply affect the interests of the Institution (such as the emergence of a new structural material or technique), the Council appoints an Ad Hoc Committee to deal as exhaustively as may be necessary with this one specific problem. Some of the greatest achievements of the Institution have been obtained in this way, and further particulars of some of these achievements will follow later in this Address.

In the meanwhile let us consider the vital part which the Branches play in the organization and adminis-

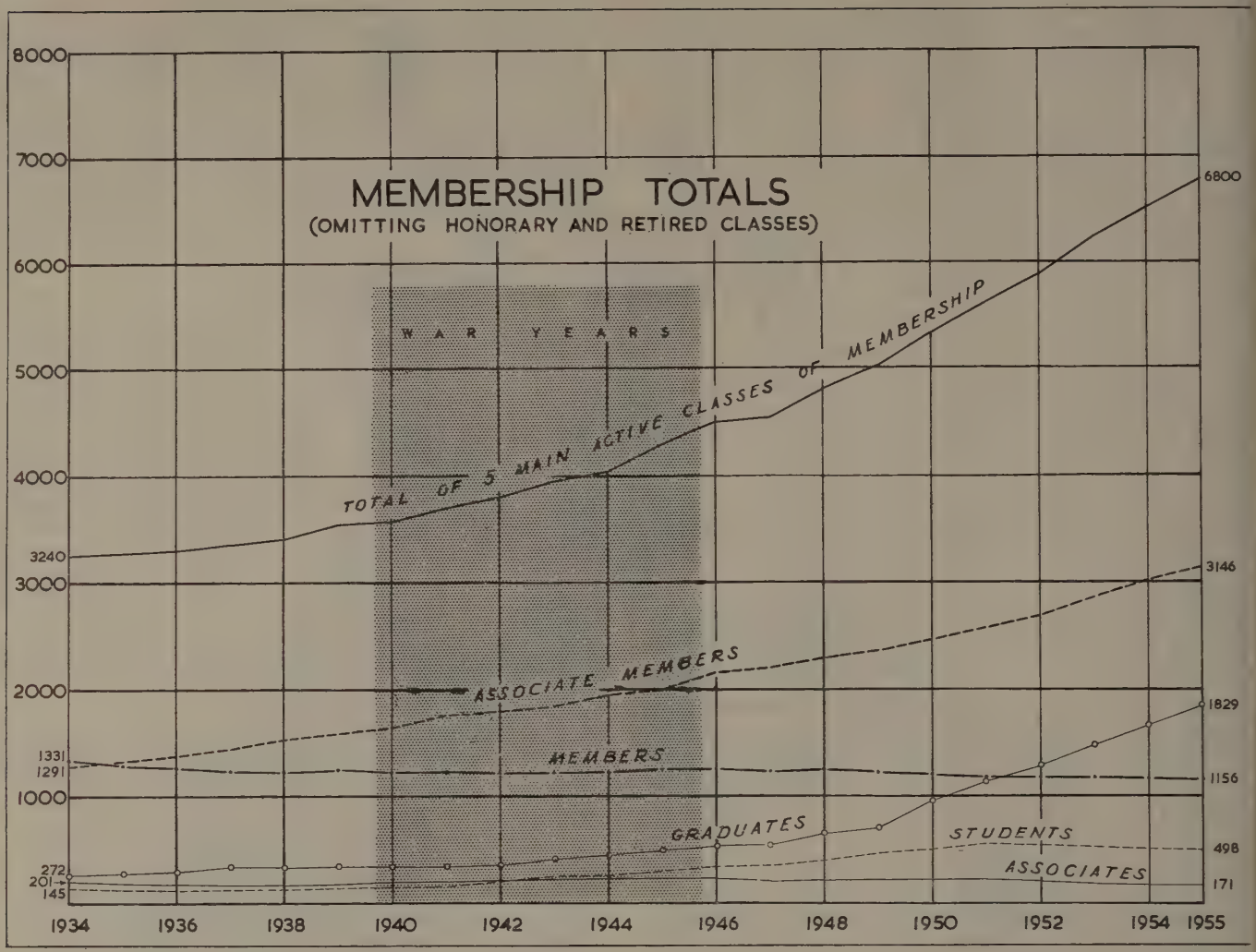


Fig. 2

tration of the Institution. In the first place each Branch Committee with its Chairman, Vice-Chairman and Honorary Secretary deals with the Institution business of the Branch, including arrangements for the holding of regular meetings for the presentation of 'papers' and for the discussion of problems which arise in the Branch area.

In addition to running its own affairs in this way, each Branch plays its full part in helping to run the affairs of the Institution as a whole. Each Branch has direct representation on the Council, the representative being the Chairman of the Branch for the year in question. In addition, three of the twelve full Members of Council and three of the six Associate-Members of Council must be country members. Every Branch representative is therefore able to express the view of the Branch on every question which comes before the Council, whether on matters of policy, finance or any other subject. Each Branch is also encouraged to bring before the Council for deliberation any matter which it considers advisable to raise, either in the interests of the Branch itself or of the Institution as a whole (which usually comes to the same thing).

There is no doubt that the growing strength of the Institution is due in no small measure to the activity of the Branches and to the energy and enthusiasm o the members of the Branches and in particular, the Branch Chairmen and Vice-Chairmen, Honorary Secretaries and members of the Branch Committees We in London do not overlook the fact that whereas a Council Meeting or a Committee meeting means t London members giving a few hours of their time Delegate Members have to get to London and back i addition and this may well mean either one or tw nights in a train.

There is one further matter relating to the grea contribution which the Branches make to the well being of the Institution as a whole. It will be under stood that even with an active and zealous Branch and Branch Delegate, it is essential that there must b good relationship between the London Headquarter and the Branch in order to achieve the high degree o co-operation aimed at.

The close friendship which does in fact exist betwee London Headquarters and the Branches is therefor not only a very pleasant thing to record, but it en hances the valuable contributions which the Branche make to the general good of the Institution.

5. Growing Numerical Strength

Let us now examine the evidence relating to th

development of the Institution during the 21 years as a Chartered body. The first thing to consider is the variation in numerical membership during this period, and this is shown clearly in Fig. 2.

In this figure are shown curves giving the variation in numerical strength of each of the five main classes of membership, namely, Members, Associates, Associate-Members, Graduates and Students, and in addition the top curve shows the variations in the totals of these five classes. No curves are given for the remaining five classes of members, namely Honorary Members, Honorary Associates, Retired Members, Retired Associates and Retired Associate-Members, as although many of these members have rendered valuable services in the past, their present and future services to the Institution must necessarily be small, and any variation in numerical strength upwards or downwards among these classes would throw no light on the picture I am trying to present of the growth of the active elements in the Institution, and would in fact confuse this issue.

If we study Fig. 2, we see that both the Membership and the Associateship totals show a small but gradual fall between 1934 and 1955, Membership totals reducing from 1,331 to 1,156 and Associateship totals from 211 to 171 in the 25 year period.

Neither of these decreases need, I submit, cause us concern. A very considerable proportion of Members nowadays acquire that status by transferring from Associate-Membership, and these must have been engaged, after qualifying for Associate-Membership, for at least five years in a position of responsibility for the design or execution of important structural engineering work. It is in fact a relatively rare occurrence for an application for transfer to be received in the minimum five year period after admission to Associate-Membership, and the average is probably about eight years. We might therefore expect the number of transfers from Associate-Membership to Membership to follow the general trend of variations in the Associate-Membership figure with a 'lag' of about eight years. The Associate-Membership curve is a rising one, so that if full membership were only obtainable by transfer from Associate-Membership, we might, therefore expect the Membership curve to show an increase instead of a small decrease over the Charter period.

There is, however, another factor operating, namely that full Membership can be granted not only by transfer from Associate-Membership but also by "direct admission," subject to the Council being satisfied that the applicant complies with the Bye-Law requirements governing such 'direct entry.' These requirements have become increasingly more onerous and consequently there has been a considerable fall, over the Charter years, in the number of admissions to full Membership by 'direct entry' and this fall has slightly exceeded the gradual increase in the number of transfers from Associate-Membership.

It seems unlikely that the Bye-Law requirements governing 'direct entry' will be made even more onerous than they are today. The fall in the numbers of 'direct entrants' to full Membership is therefore likely to stop in the near future. On the other hand the number of transfers to full Membership from Associate-Membership is likely to continue increasing (since it should follow approximately and with a lag of some eight years or so the steadily increasing numbers of Associate Members.)

It seems clear therefore that we may confidently expect the gradual fall in the total numbers of full Members to cease in the near future and then to change to a steady and continuing increase.

With regard to Associates, it is quite natural that the curve of numerical strength should follow, as it does, the general trend of that for Members, when we remember that Associates are persons of not less than 43 years of age and who are engaged in professions allied or kindred to the profession of structural engineering and who satisfy the Council that they are of a professional standing equivalent to that of Members, and who are deemed worthy of Associateship.

The real check on the health of the Institution, however, lies in the figures for Associate-Membership, Graduateship and Studentship, as the well-being of the Institution in the near future will rest largely in the hands of Associate-Members, and in the more distant future in the hands of the Graduates and Students in that order.

Let us first consider the Associate-Membership curve. It will be seen that it is very reassuring, showing a healthy and considerable growth, at a gradually increasing rate, the total increase from 1934 to 1955 being from 1,291 to 3,146, nearly $2\frac{1}{2}$ times as great.

The Graduateship curve is rather different in form but is at least equally encouraging. The number of graduates remained almost constant from 1934 to 1942 and then increased slowly but steadily until 1947. Then the rate of increase went up very considerably and this high rate has been maintained with remarkable steadiness up to the present day. The total number of Graduates has increased from 272 in 1934 to no less than 1,829 in 1955, more than sevenfold.

The curve for Students shows that their numbers remained fairly constant from 1934 to 1946 at round about 145, and then started increasing, and this increase continued at a steady and very satisfactory rate until 1951, when the figure stood at 529. Since then the increase has stopped and there has been a slight fall, the latest figure for 1954 being 498. This slight decrease in the number of students (31 in four years) is at first sight a little disappointing but examination reveals that the number of transfers from Studentship to Graduateship is increasing and this appears to outweigh at present the number of 'new' Students. It is also believed, though this is more difficult to check, that an increasing number of young structural engineers may be endeavouring to become Graduates direct, without becoming Students first and before they are ready for Graduateship.

If we now look at the top curve which shows the totals of all the five active classes which we have so far considered individually, we see that at the beginning of the Charter period there was a slow but steady increase up to the beginning of the Second World War. During the war the figures remained approximately constant, but soon after the war the numbers started increasing at a considerably greater rate than the pre-war rate, and (with the exception of one particular year, 1947) has continued with quite remarkable and healthy steadiness up to the present time, the total increase during the Charter period being from 3,240 in 1934 to 6,800 in 1955.

To summarise, therefore, I would submit to you that these figures, together with the analyses of the various individual curves, reveals a surprisingly regular and

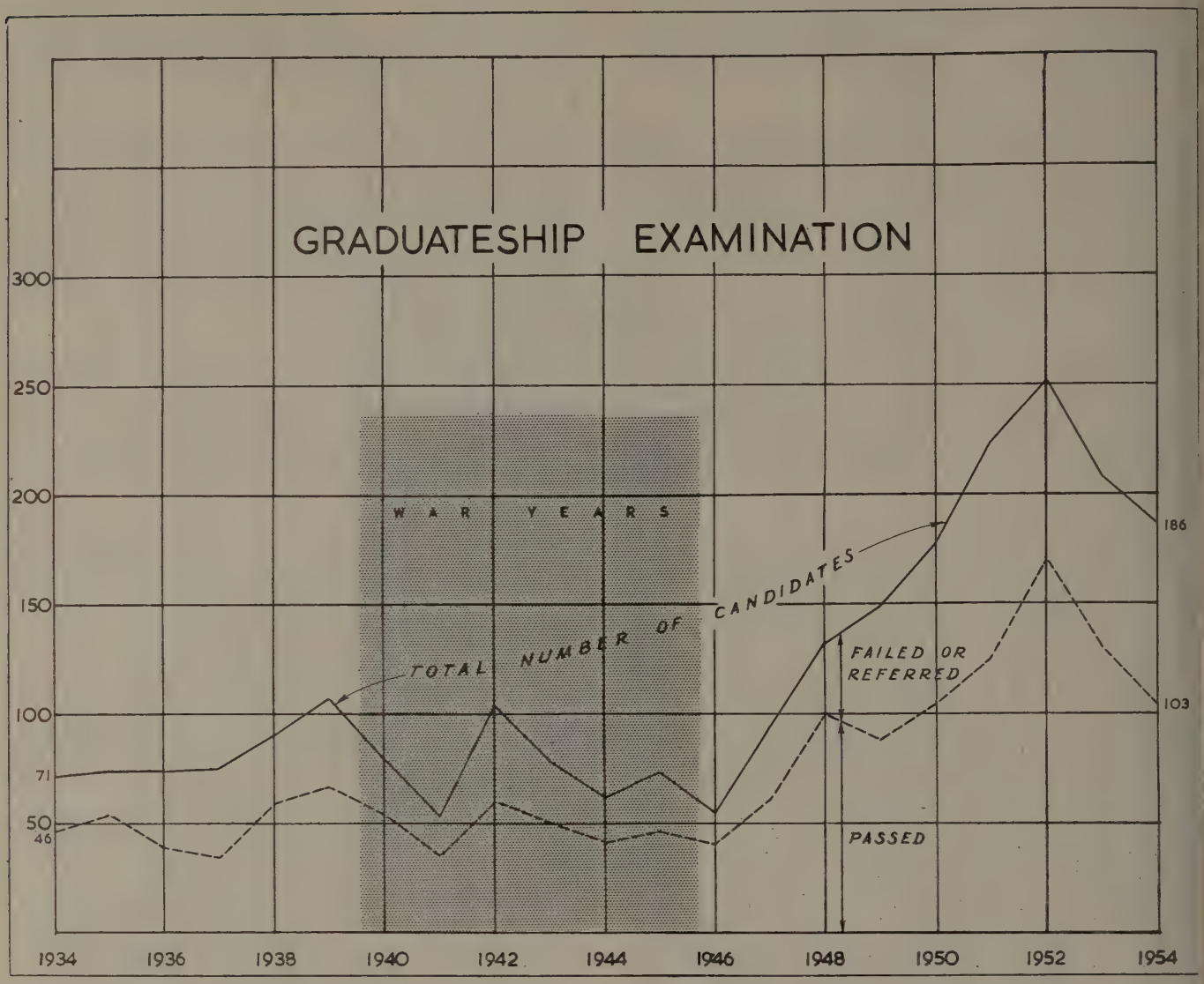


Fig. 3

healthy numerical growth, with every indication that this will continue, as there is practically no " bending over " of the " Total " curve, as must eventually be expected when saturation point approaches.

So much for the quantitative review.

This would be of little importance unless we can be satisfied that there has been no falling off in quality. Let us then consider this question of " quality."

6. Conditions of Entry and Institution Examination

Admission into the Studentship, Graduateship and Associate-Membership classes is by examination only, transfer from a lower to a higher grade being encouraged, subject to the applicant passing the Institution's Examination in the required subjects of the higher grade. Examinations of certain other bodies have been recognised after most careful scrutiny, as being of such a standard that the Institution grants exemption to those who have passed such examinations from part or whole of the Institution's Graduateship Examination, and also from part of the Associate-Membership Examinations.

In the case of Studentship, the examination which is a test of general education is shared with several other Engineering Institutions, and is known as the Common Preliminary Examination.

In the case of Graduateship and Associate-Membership, the Institution sets its own examinations.

In the case of full Membership admission is granted in two ways, subject in both cases to the detailed requirements of the Bye-Laws.

(a) By transfer from Associate-Membership.

(b) by direct entry without examination.

The job of ensuring that a high standard of entry into the active grades of membership intended by the above provisions is maintained, is carried out by two of the Standing Committees, subject always to the approval of the Council.

In the first place the Education and Examinations Committee draw up the syllabuses of the Graduateship and Associate-Membership examination, and arrange for the setting and marking of the papers and report the results.

The Membership Committee scrutinise every application to each grade of membership and make their recommendations to the Council.

The recommended lists are considered in Council and only after approval by Council is an applicant granted admission to the appropriate class of membership.

Nevertheless the question as to whether the standard of entry by examination into the appropriate grade of membership is being maintained is ultimately dependent on the standard of setting and marking of the examination papers. A comparative study of papers in recent years with those set in the early Charter years will, I think, convince any one that the standard of knowledge and ability required to obtain a " Pass " has definitely risen.

It may, however, be of interest to consider the figures for the total number of candidates for the Institution examinations and the total number of " Passes."

These are shown in graphical form in Fig. 3 for the Graduateship Examination. The upper curve gives the total number of candidates and the lower curve the number of successful candidates. It will be seen that until about 1946, both the number of candidates and the number of " Passes " remained fairly constant (except for some fluctuations during the war years). A very considerable increase in the number of candidates then started and continued at a rapid and steady rate until 1952. The number of successful candidates increased in a similar way but rather less rapidly, indicating either that there was a growing tendency for candidates to take the examinations before they were really ready to do so, or that there has been a stiffening up of the standard required.

From 1952 to 1954 the number of candidates has fallen appreciably (from a peak of 251 in 1952 to 186 in 1954) and of course the number of " passes " has dropped in consequence. The reason for this fall is a matter to which further consideration will have to be given by the Institution should it continue.

It will be seen that notwithstanding the drop in the last two years, the total increase in number of candidates for the Graduateship Examination between 1934 and 1954 was from 71 to 186 and the number of ' passes " from 46 to 103. The percentage of passes has fallen from about 65% in 1934 to about 55% in 1954.

Before leaving the subject of the Graduateship Examination it should be added that a completely revised syllabus is being drawn up by the Education and Examinations Committee for submission to the Council and no comment can therefore be made about this while the matter is still sub-judice.

The Associate-Membership Examination is intended to be a test of a candidate's knowledge of the principles of structural engineering design and of his ability to apply these principles to practical design problems. It requires a standard in these subjects at least equal to University degree standard. A particularly valuable feature of the examination is the Design paper in which optional alternative design problems are set. Candidates are allowed to bring into the examination room any books which they think may be helpful, so as to reproduce as nearly as practicable, the conditions which would apply in an engineering office. This particular examination paper is not intended to test the candidate's memory, nor his theoretical knowledge of the principles of structural engineering, but his skill in selecting a suitable solution for a design problem and in developing this design by making such calculations and detailed drawings as may be called for.

May I emphasize that although certain University degrees and other equivalent examinations are accepted as granting partial exemption from the Associate-Membership Examination, all applicants without exception are required to sit for this Design paper.

Fig. 4 shows the curves of total candidates and successful candidates respectively for the Associate-Membership Examination through the Charter period. In 1934 there were 100 candidates of whom 65 were successful. From 1934 to 1946 there was a general tendency for both curves to rise, subject to rather violent fluctuation during and immediately after the war.

In 1946 there were 148 candidates of whom 81 were successful (about 55%). Then in 1946 a very considerable increase in numbers of candidates started, and this increase has continued at a remarkably constant rate year by year until 1954 (the last year for which figures are available). The number of successful candidates showed a similar but less rapid growth up to the year 1952. During this period the percentage of " passes " reduced steadily from 55% in 1946 to 38% in 1952 indicating either (a) a falling off in the state of preparation of the candidates, or (b) a stiffening up of the standard of the questions set or of the standard required to achieve a " pass " or perhaps a combination of (a) and (b).

From 1952 to 1954, in spite of a steadily growing number of candidates, the number of successful candidates has reduced considerably. The reason for the sudden fall in the percentage of passes in the period 1952 to 1954 compared with the period just before 1952 is obscure, and is receiving the attention of the Institution.

Reviewing the foregoing analyses of the Institution's Examinations as a whole, all the evidence shows that far from there being any relaxation of standard of entry by examination into the Graduateship and Associate-Membership grades of the Institution, there has been a definite raising of standard.

Moreover, in the case of full Membership, to which ' direct entry ' is available (in addition to the usual course of transfer from Associate-Membership) the conditions of such ' direct entry ' are considerably more onerous now than they were at the beginning of the Charter period.

We are now therefore in a position to draw the conclusion that not only has there been a healthy, steady and surging growth in the numerical strength of the Institution, but the " quality " of the various classes of membership has not only been maintained, but has been enhanced through the Charter period.

7. Activities and Achievements

The activities and achievements of the Institution during the Charter years can best be appreciated by considering them in relation to the very remarkable developments which there have been in the field of structural engineering during this period.

It may be difficult on the one hand for young engineers to realise how very much more restricted was the field of structural engineering 21 years ago, and similarly it may be difficult for ' non-members ' to realise how great and widespread is the present day scope of structural engineering.

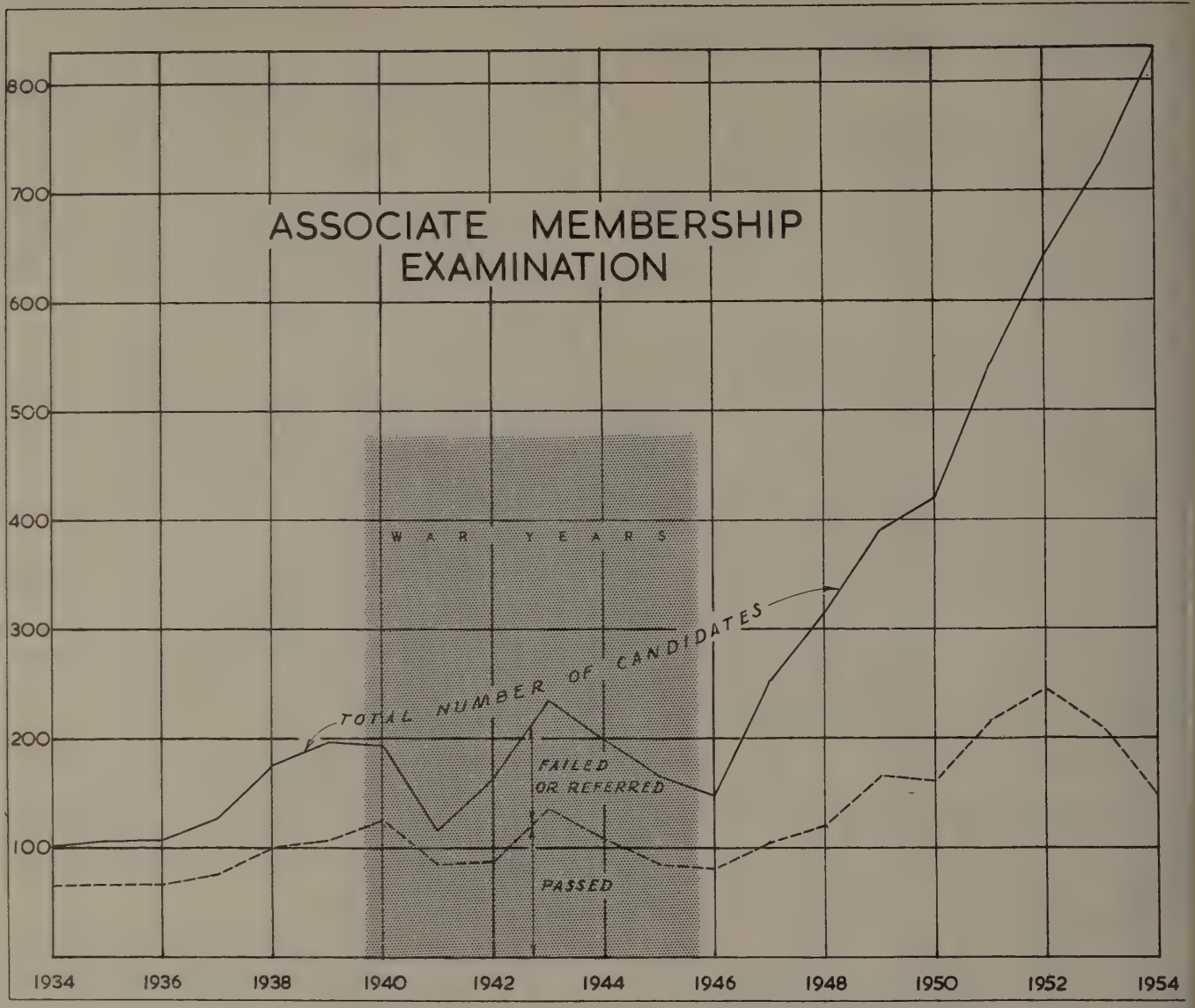


Fig. 4

At the beginning of the Charter period, for example, welding applied to structural steelwork was in its very infancy, aluminium alloy was not considered as a structural material, prestressed concrete was something we had heard about but which I regret to say we, in this country, had done nothing about, soil mechanics was only beginning to emerge from the theoretical into the practical field, vibration of concrete was little understood and rarely used. Shell construction was unknown or at any rate unused in this country, concrete mix design had not got beyond the most elementary stage, the moment distribution method of solving elastic rigid frame problems had only recently been introduced, design in accordance with the plastic theory was unknown. Each of these structural engineering matters (and many others) is now a science of its own with its own bibliography and in most cases with research workers engaged full time on the particular item in question.

What part has the Institution taken in these great advances in Structural Engineering in the last 21 years? One of the vital steps the Institution took was to help to lighten the dead hand of the restrictive regulations which in 1934 governed structural engineering design and which had a blighting effect on any new developments.

The Institution sponsored the committees whic culminated in the publication of the Code of Practic (CP113) for the Design of Structural Steelwork fo Buildings and the Code of Practice (CP 114) for th Design of Reinforced Concrete in Buildings and oth structures.

It is true that British Standard B.S. 449 was pul lished very shortly after the publication of CP 11 but B.S. 449 appears to have been based on CP 11 and only differs from it in relatively minor matter and most authorities accept designs prepared in accor ance with the provisions of either of these document Since the publication of these documents the Londo County Council has completely revised its Bye-law and care was obviously taken in the redrafting of the Bye-laws to make them conform as closely as possib with these two Codes of Practice. Revised Model Bye-law in turn have been based on the revised L.C.C. Bye-law

These measures meant that Structural Engineerin was released from the fetters of restrictive regulatio by which it had been bound for so long (although the had been some intermediate loosening of these fetter This freedom has resulted as one might expect in gre advances all along the line. Perhaps the most im portant of all the new provisions is the one whic allows for the application of new techniques an

methods of design not known at the time of drafting the Codes, but which might be evolved later, thus encouraging the development of new and original ideas.

Apart from its successful efforts in freeing Structural Engineering from the shackles which bound it for so long, the Institution has drawn up and published reports on many subjects of primary importance to Structural Engineers. Two examples of Reports on new developments are :

(a) Report on the Structural Uses of Aluminium Alloy and

(b) Report on Prestressed Concrete.

These comprehensive Reports gave the first authentic guidance available in these two entirely new fields of structural engineering design (apart from what could be gleaned from text books and published papers presented to learned Societies and Institutions) and they have proved invaluable to members of this Institution and others. There can be little doubt that these Reports have contributed very materially to the widespread development in these two new fields of Structural Engineering. In each case the new technique which has developed has virtually placed a new structural material in the hands of the Structural Engineer.

Other Institution Reports and Publications deal with subjects which have long been the concern of Structural Engineers but in which it was felt either that guidance was needed, or that general practice was outdated. In such cases after exhaustive study of the subject, recommendations for practical application have been formulated in line with the latest results in all parts of the world.

A valuable example of this kind is the Code of Practice for Earth Retaining Structures which was issued by the Institution in 1951 on behalf of the Civil Engineering Codes of Practice Committee.

Time does not permit me to deal with the many other valuable Reports and Publications which have given guidance to our members and others, in many branches of our profession, but a complete list of these Reports is given in the Institution's Year Book including Joint Reports with the Institution of Civil Engineers.

In addition our members have served on and are serving on a great number of British Standard Committees and other Joint Committees with other Institutions covering a great variety of subjects pertaining to Structural Engineering and in this way our members are actively engaged in promoting the general advancement of the science of Structural Engineering, which is of course one of the main objects and purposes of our Institution as prescribed in our Charter.

A complete list of all the Committees on which we are represented will be found in the Institution's Year Book and it is an impressive list.

8. The Art of Structural Engineering

This may be an appropriate place to emphasize the fact that our Charter refers to the advancement of the science and *art* of Structural Engineering.

There is no doubt about the immense advances along so many lines of the *science* of Structural Engineering and the large part which this Institution has played in these scientific advances, our members having been in the forefront of research in all fields of Structural Engineering.

But what progress has been made in the " art " of Structural Engineering ?

We should here, I submit, distinguish between two classes of structure :

Class (a) Structures such as steel or reinforced concrete framed buildings, in which the framework is going to be more or less concealed.

There seems little doubt that in general, the most successful buildings of this class are those in which an Architect is appointed for the building and a Structural Engineer is appointed *in the earliest possible stages* to collaborate and advise on the structural problems involved. (In many cases the earliest possible appointment of an Engineer to advise on the heating, ventilating and other problems is also very desirable, so that all requirements can be considered and co-ordinated in the very earliest stages of design).

In such cases, the Architect is of course responsible, amongst other things, for the conception of the building as a whole, and for all the aesthetic problems which arise. The Structural Engineer's first duty is then to examine the Architect's sketch plans from the structural angle and recommend and discuss with the Architect any modifications which may seem advisable, should the original sketch plans show any particularly difficult structural problems or problems involving excessive expenditure for their solution. After general agreement is reached on the form and arrangement of the building, the Structural Engineer's duty is then to so arrange and detail the structural elements as to fit in most effectively and economically with the Architect's requirements.

We, as Structural Engineers, know how much more successful such collaboration is when we are working with Architects who have clear understanding of the structural problems involved and of our proposals for their solution.

I believe it is also of the greatest value to successful collaboration if the Structural Engineer is, for his part, sufficiently knowledgeable in aesthetic matters, to have a clear understanding of the aesthetic problems which confront the Architect and of his proposed solutions of these problems.

Class (b) Structures in which the structural elements remain unclad or in which they provide the chief determining factors for the form of the structure as a whole.

In these cases aesthetic considerations must be taken into the fullest account by the designer right from the earliest stages of the design when the general form and proportions of the structure are being developed, right through to the design of the smallest details.

The question as to what constitutes an aesthetically satisfying structure is a great and contentious question, but nevertheless I am going to venture to make a few personal and elementary comments on this subject in order to stimulate interest of members in this important part of the ' purposes ' of the Institution as laid down in the Charter. I am emboldened to do this in the sure knowledge that whereas a mathematical statement, (and generally also a scientific statement) can be checked, and if erroneous can be proved to be so,

no one seems able to prove that a statement on any subject of art is wrong ! The most that can be said by even the most eminent or violent critic is that he does not agree with the statement (though it is true that this can be said with various degrees of emphasis and politeness).

It is my belief that the laws of nature are such that when mathematically correctly applied to give a structure its required strength without waste, and without compromise to meet other requirements, the resulting structure will be satisfying to the eye. It may even be that the sub-conscious mind realises the ' perfect form ' which mathematical application of the laws of nature would call for and the eye is satisfied in proportion to the nearness which the actual design approaches to the subconsciously perceived perfect form.

If this were the whole truth it would seem that a well designed structure from the mathematical and scientific viewpoint would also be an aesthetically satisfying structure.

Good engineering design however, is necessarily something much more complicated than the solid embodiment of pure mathematics. Many other requirements have to receive full consideration by the engineer (including the very obvious limitations imposed by practical methods of fabrication). These other considerations obviously complicate and confuse the issue of creating a pleasing structural form and can lead to anything but aesthetically satisfying structures unless careful thought and good judgment are applied to the aesthetic problems involved.

Furthermore the approach to what may be called the ' perfect mathematical form,' and the aesthetically satisfactory adjustments necessary to meet requirements other than the mathematical ones, form only part of the aesthetic problems which face the designer. Other factors must receive full consideration to achieve an aesthetically satisfying result, for example, surface texture and colour (especially in relation to the surroundings) and the very contentious question of the use of decoration, (that is to say the addition of some features not structurally essential), perhaps for the purpose of emphasizing particular features of the structure, or of providing a focus of interest.

It is therefore clear that many of the aesthetic problems must be solved conjointly with the mathematical and other problems, in order to produce a structure which is satisfying to the eye as well as being efficient for its purpose. There is little doubt that modern scientific developments in Structural Engineering and in fabrication techniques have made available to the engineer a far greater choice of form than was formerly the case, and this great freedom of choice clearly makes it all the more important that constructive imagination is used in dealing with the design of structures in this class, and that the various aesthetic requirements do receive full consideration.

The increasing number of structures in this class which have been built in recent years which are generally accepted as being ' aesthetically satisfying,' shows the growing attention which is being given to the promotion of the ' art ' as well as the science of structural engineering. But we must all admit that there is still a long way to go along this road.

Universities and Technical Colleges are now beginning to draw attention, and in some cases give instruction on the aesthetic consideration of engineering design and it is hoped that this development will grow into general and regular adoption. But the development of good taste cannot be achieved just by attending a course of lectures on aesthetics, valuable as this may be. Good taste is generally the product of a widely educated mind and young engineers are recommended to extend the general scope of their reading and study, to cover as wide a range of subjects as possible, and also to study at all times as great a variety of structures as possible, particularly those which give general aesthetic satisfaction. Where possible this study should be of the actual structures, but often this is impracticable and the next best thing is then to study photographs of structures published in various publications, and this should include structures from all parts of the world. A regular and prolonged comparative study of a great variety of fine structures in this way should help to produce a state of mind from which will grow a style which is personal to the individual in question, and when this development of personal styles becomes widespread, it is likely to result in an increasing variety of aesthetically satisfactory structures.

While there seems to be little doubt that " good taste " flourishes and develops best in a widely educated mind, the great gift of " constructive imagination " would appear to depend little if at all on education, and it is in fact sometimes displayed by young children and by entirely untutored " savages." The possession of this creative gift in a high degree is however comparatively rare.

Most of us would therefore do well to extend the scope of our education (and the comparative study of aesthetically satisfying structures) in the hope that even though the spark of genius may not be ours, we may nevertheless develop ' good taste ' to a sufficient degree to enable us to produce structures which satisfy the eye.

9. Status

So far we have considered material evidence of the vigorous growth of the Institution and also its activities and achievements during the Charter years. Let us now consider the less tangible but very important and delicate question of ' status.' This is incapable of being expressed in numbers and depicted in the form of a graph, but it can be considered qualitatively by the examination of appropriate evidence.

In the first place what exactly do we mean by ' status ? ' The status of an Institution is, I submit, a measure of the recognition and esteem in which an Institution is held by the general public and in particular by its fellow Institutions.

Let us consider then whether in the case of the Institution of Structural Engineers this ' recognition ' is in line with its actual activities and achievements.

I have already outlined earlier in this address some of the outstanding work which has been and is being carried out by the Institution, and which has earned widespread recognition. There is much other evidence that a growing and world wide recognition is being accorded to the Institution.

For example the Institution's Journal " The Structural Engineer " has established a reputation which has resulted in greatly increased orders coming in from all parts of the world ; and the papers published in

it are more and more frequently quoted in engineering literature.

Further evidence of the growing world-wide influence of the Institution is provided by the fact that we now hold examinations (twice yearly) on an average at 35 centres outside Great Britain, and these centres are not limited to the Commonwealth.

Furthermore, the remarkable increase in the numerical strength of the Institution during the Charter years (and the much greater increase in the number of applications for all classes of membership) is itself an indication of the great growth in the prestige of the Institution.

It is however inevitable during a period of rapidly growing strength and influence of a young Institution, that there must be a lag between the increasing volume of achievements and the recognition of these achievements by the general public and by fellow Institutions. This is quite natural and understandable.

Even the most unprejudiced 'outside' minds can, in all sincerity, be likely to base their view on the status of a young Institution like ours on the opinion which they formed of it when it was in an earlier state of development, unless indeed they are made aware of the up-to-date facts. Perhaps a word of warning is advisable here against excessive zeal leading to 'overplaying' one's hand in this delicate matter, as this would be both undesirable and unnecessary.

For example the recognised "senior engineering Institution" of this country has earned universal and world-wide reputation both for the immense scope of its activities and for the distinguished services which it has rendered for so long (and which it is still rendering) to British engineering, and it is not our wish to encroach in any way on the great prestige which has been so well earned by this fellow Institution. But in the case of the Institution of Structural Engineers our full energies are concentrated on the necessarily narrower field of Structural Engineering, and we feel that in such matters the Institution has now proved its worth. We have for many years co-operated on terms of equality with the "senior engineering Institution" in many matters concerning our joint interests, and we have taken the lead when such matters have been primarily the concern of Structural Engineers, and we are ready and able to do this at any time.

Recognition of this position is now generally afforded, but when we occasionally come across cases where this is not so (generally through lack of understanding of the true position) it is the duty of the Institution as a whole and of each individual member of it to make the true facts known.

10. Conclusion

In this address it has not of course been possible to do more than sketch in the main outlines of the 'pictures' which I wished to present to you.

I believe, however, that the picture which has emerged shows the Institution of Structural Engineers to be an active, vigorous and steadily growing Institution, which keeps closely in touch with the latest developments in Structural Engineering, and assists in making such research work available for application to actual structures, and which throughout the Charter period has made and is continuing to make great advances along the road set out for it in the Charter.

I submit therefore that now that the Institution of Structural Engineers has reached its coming of age as a Chartered body, it has grown to the stature of full adult brotherhood with the other great but longer established engineering Institutions of this country.

The Charter years, which have seen such remarkable developments in many directions in Structural Engineering have put the Institution of Structural Engineers to the test and it has not been found wanting.

Whilst, however, we can all feel proud of the growth in strength, achievements and status of the Institution during the Charter period, there can be no room for complacency. It must be remembered that the personnel of all Institutions must be gradually but continually changing and continued progress towards the great future which I believe lies before us, must therefore depend in large measure on the energy, ability, and above all, the enthusiasm which the younger members are prepared to give to it.

Acknowledgment

In conclusion I wish to record my very grateful thanks to our Secretary, Major Maitland, for kindly providing me with all the facts and figures for which I asked him, to enable me to prepare this Address, and subsequently, at my request, for reading through my initial draft, and offering me his very valuable comments and criticisms which resulted in my making many variations to the original draft.

The Tignes Hydro-Electric Scheme

By R. Giguet

Ingénieur et Directeur de l'Equipement, Electricité de France

Synopsis

DESCRIPTION of an important development in the French Alps, harnessing the upper section of the river Isère. Its main parts are a high concrete dam at Tignes, and two power stations at Brévières and Malgovert.

After general data on the site and its hydrology, the concrete dam and its construction works are described briefly; then following a short reference to the Brévières power-plant, information is given on some of the most important, new or original achievements related to the Malgovert power station: the drilling of the head race gallery through the slopes of the "Mont Pourri" mountain range, and the assembly of two powerful penstocks.

The second part is devoted to the Malgovert station, its equipment with four 80 MVA generating sets, and some of the new problems which had to be solved to design, build and transport such large generators. The three problems selected for a short report are the generator rotor construction, the bearings construction, and the generator transports from the factory workshops to site at Malgovert.

The Tignes Hydro-Electric Scheme

Among the recent achievements of E.D.F., the Tignes scheme presents many very interesting features and has been thought worthy to be presented before this meeting.

Since this meeting is interested both in civil engineering and electro-mechanics, this short lecture will be divided in two approximately equal parts dealing with these two aspects of the question. It will be followed by a sound film on the dam construction work.

First Part
—General Layout and Civil Engineering

Tignes is located in the French Alps, on the upper valley of the river Isère, about 60 miles South East of Geneva, and near the French-Italian border.

The Isère upper valley features are most favourable for a powerful hydro-electric development and are as follows:

—plenty of glacier and water reserve in the catchment area

—a particularly suitable site for the erection of a dam due to the high altitude (1,800 mètres=5,900 ft. above sea level), the narrowness of the canyon with strong rocky abutments on either side, the large capacity of the natural reservoir upstream to the dam site

—a steep slope downstream.

The counterpart to these advantages was the necessity to flood the Tignes village and to move 350 inhabitants into new dwellings.

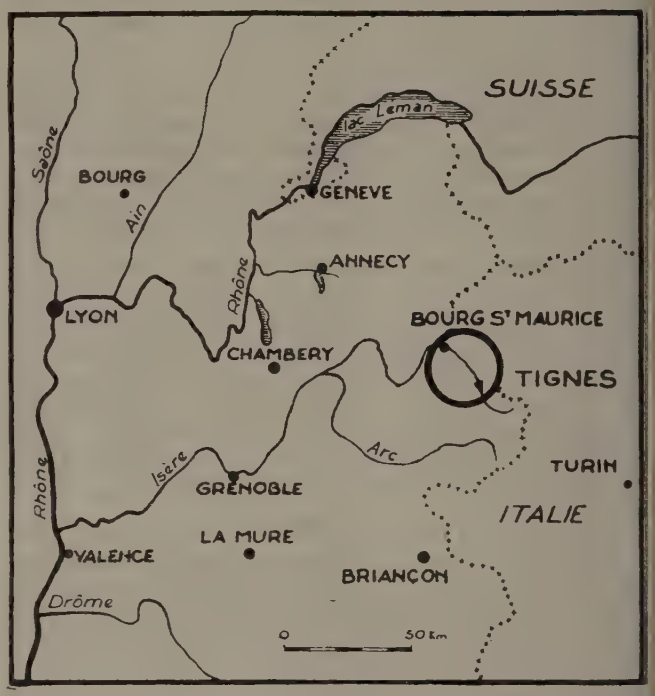


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Fig. 1.—Map showing the location of the project

The Tignes scheme is made up of three main parts

—The *Tignes* concrete dam and its reservoir

—The *Brévières* hydro-electric plant located 1 km. (0.625 mile) downstream from the foot of the dam and using the head created by the dam

—The *Malgovert* hydro-electric station 17 km. (10.06 miles) downstream from the Brévières plant and operating under a head of 750 mètres (820 yds.).

Put together, Brévières and Malgovert total a gross head of 983 mètres (1,075 yds.).

Besides, the natural catchment area of the river Isère itself at Tignes is augmented by the diversion of two neighbouring basins: on the right bank the waters of the torrents "des Clous" and "du Nant Cruet," and on the left bank the upper part of the "Ponturin" and "La Sachette" torrents are brought into the Tignes reservoir.

* *Paper to be read before a Joint Meeting of the Institution of Structural Engineers and the British Section of the Société des Ingénieurs Civils de France at* 11, *Upper Belgrave Street, London, S.W.*1 *on Friday,* 16*th December,* 1955 *at* 6 *p.m.*

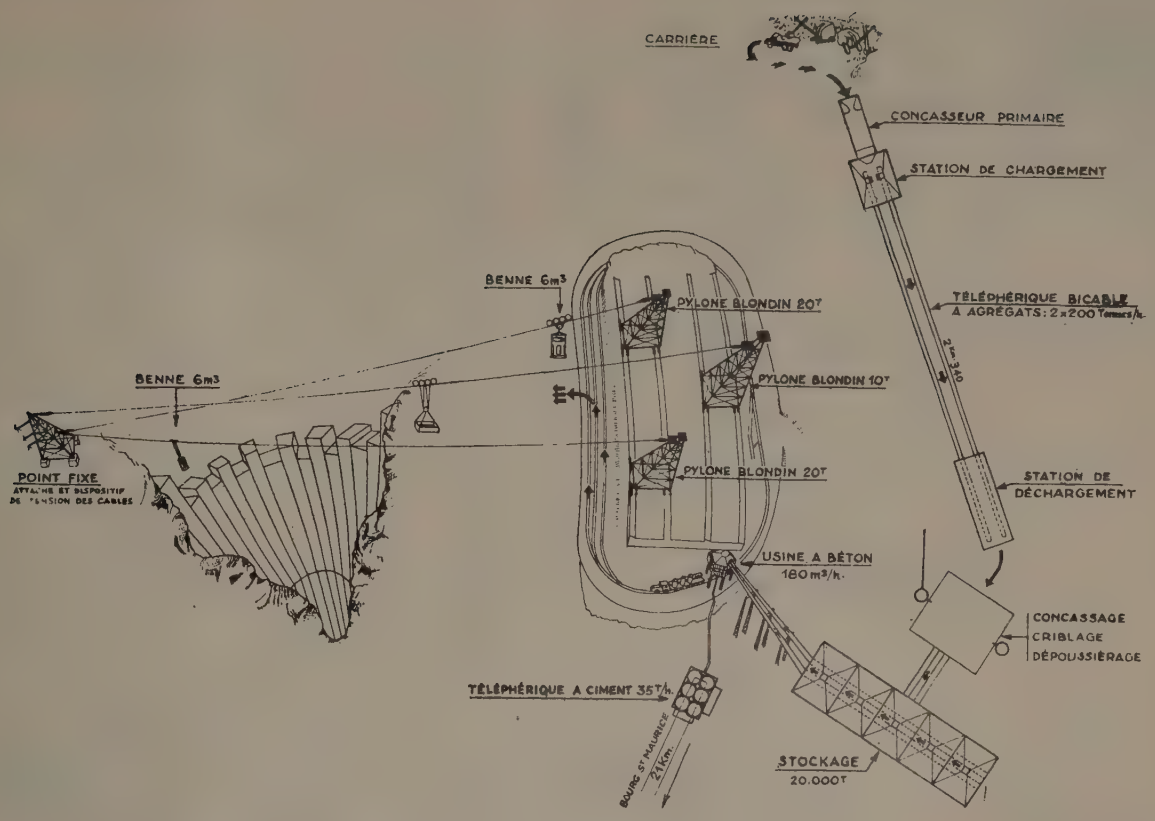


Fig. 2.—Schematic diagram of the whole scheme

The water regime is mainly glacial; low waters yields in winter with a fairly constant minimum from December to April, and high summer waters with a maximum occurring in July.

The Tignes Dam

The dam is founded on a natural defile closing the Tignes basin and made up of trias quartzite of excellent quality; these rocks are extremely water-tight.

The dam is the keystone of the Tignes development; it is 180 mètres (590 ft.) high above the bottom of the foundation and 160 mètres (525 ft.) above the original bed of the river; it is, at present, the highest dam in Europe. The chosen type, the arch dam, greatly improves the stability, it renders possible a very low value of the thickness of the foot of the dam (43 mètres =140 ft.), a reduction in the concrete volume down to 630,000 m³ (825,000 cu. yds.), and an important saving, above half the cost of a conventional gravity dam.

The upstream face takes the form of a vertical cylinder of revolution, with a 150 mètres (490 ft.) radius; the upper arches rest on two small abutment sections acting as gravity walls on each bank.

The thickness is 10 mètres (33 ft.) at the crest, and the downstream face has a curved surface such that the thickness increases up to 43.5 mètres (143 ft.) at the centre of the base (base crown).

The crest of the dam carries a road 8 mètres (26 ft.) wide at the carriage way, between two footpaths.

The water intake and the reservoir drainage are installed in the same polygonal-shaped tower, the second just under the first, at the base of the upstream face; they are provided with concrete screens.

The water intake, equipped with roller gate valves, feeds through a pressure pipe-line the Brévières power station.

The roller-gate valves of the drain are similar to the water intake ones, and like them are controlled from a control-room located near the crest of the dam; they can be lifted up to bridge structures near the dam top, for maintenance purposes.

The flood discharge at Tignes is not greater than the rated flow of the Brévières Power Station and Tignes water intake (50 m³/s.=65 cu. yd./s.); furthermore each of the two bottom drains has an output of 120 m³/s. (157 cu. yd./s.); thus, it has not been found necessary to install any surface flood weir.

A wide layout of work was displayed for the erection of the dam: the cement was transported by rail up to Bourg St. Maurice and then to the workyard by a cableway, 21 km. (13 miles) long.

Another cableway brought to the site of the dam the aggregates from the quarry, located 2 km. (1.25 miles) upstream.

The " Plateau des Boisses," on the left bank above the dam was the place selected for installation of the crushing plant, the storage silos and stock piles, the concrete plant with four concrete mixers of 3 m³ (4 cu. yds.) each.

Two 20 ton and one 10 ton cableway cranes (blondins) distributed the concrete and placed the shuttering boards.

The excavation works and the lower parts of the dam concreting began in 1948-1949 while the main civil engineering equipments were erected.

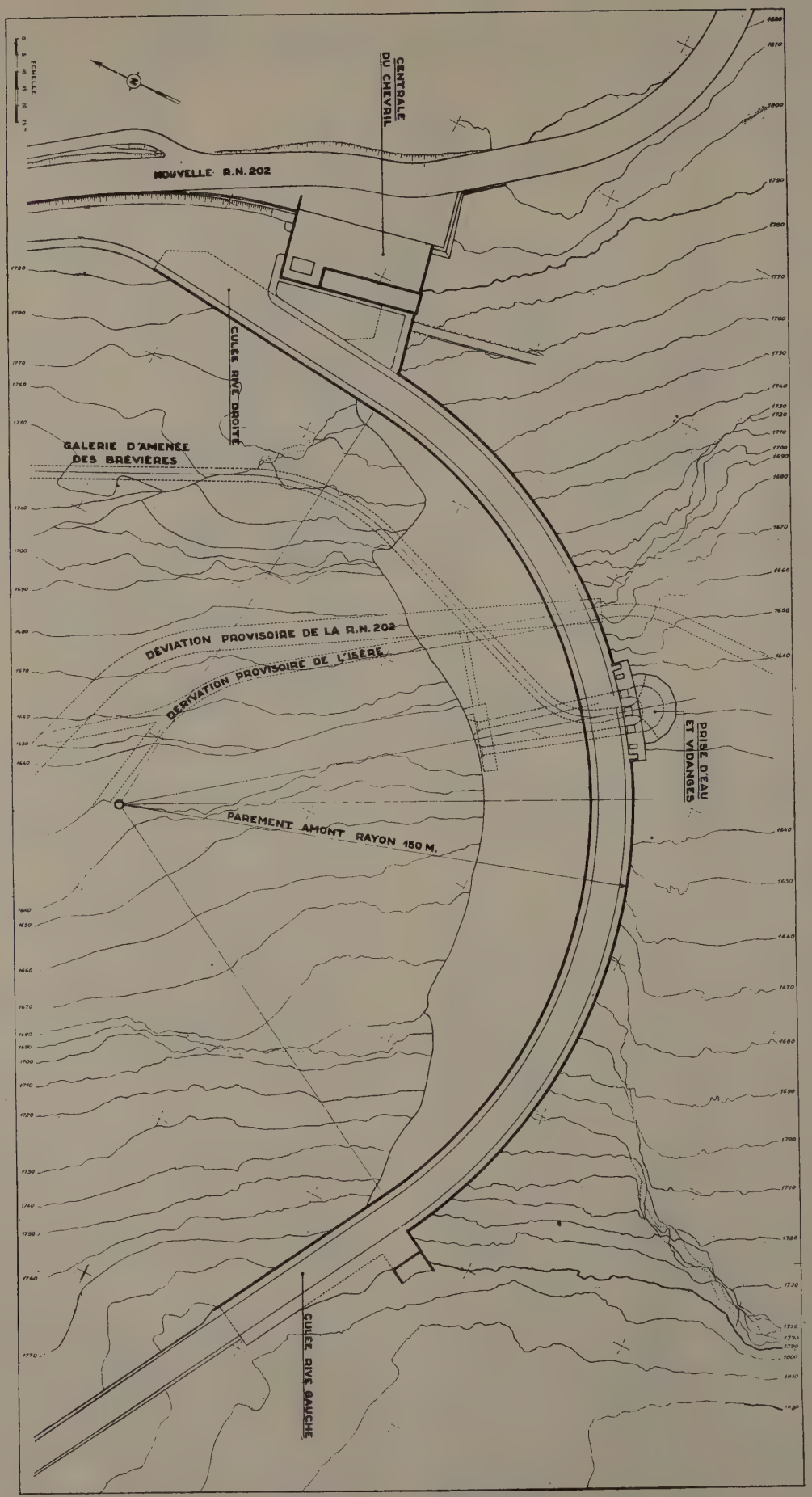


Fig. 3.—Plan view of the dam

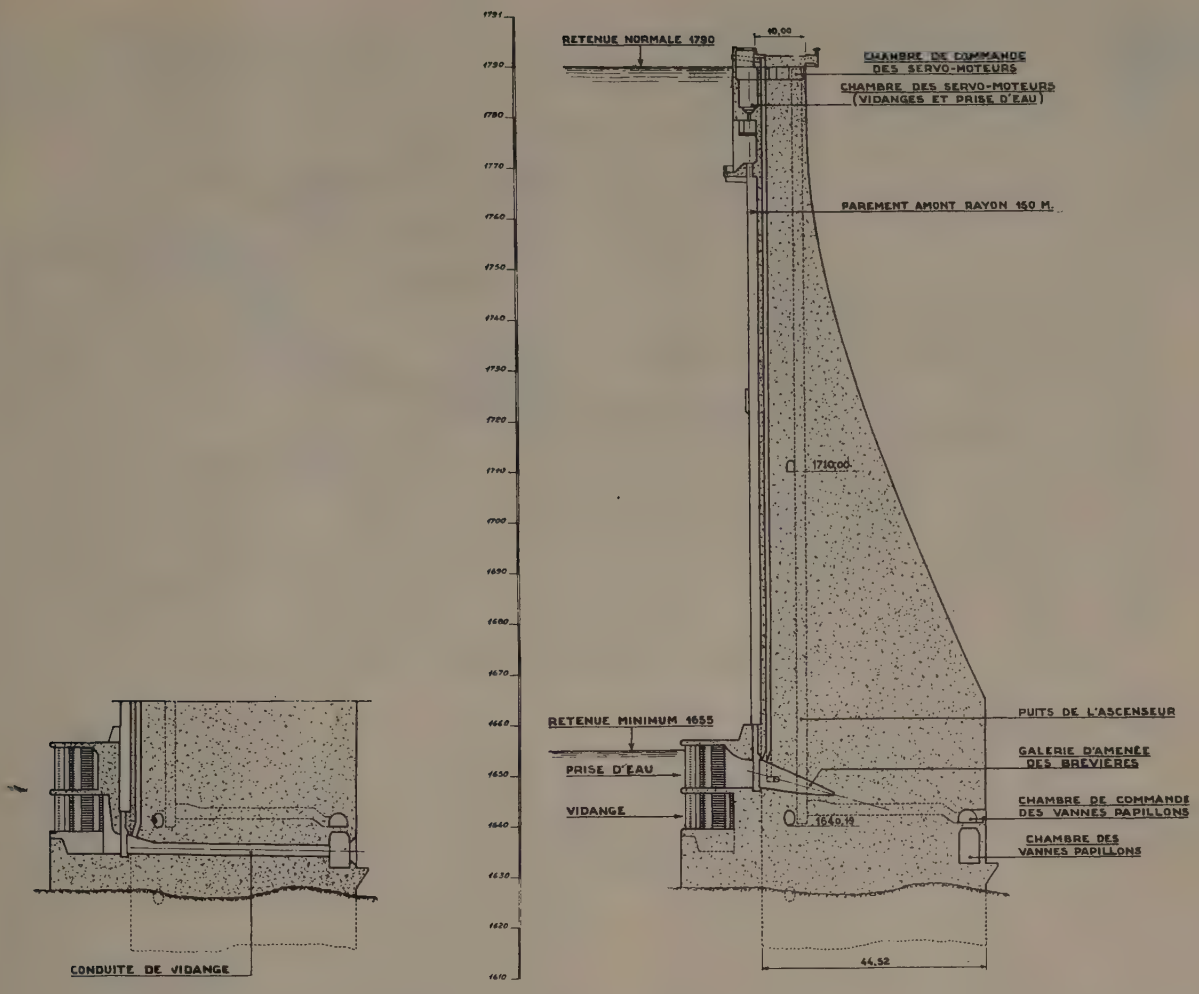


Fig. 4.—Vertical section of the dam

The erection of the dam took three summer campaigns : 1950-1951-1952. The concrete was produced with three shifts, the peak daily production was 4,500 m³ (6,000 cu. yds.) on the 7th July, 1952 : the monthly peak production was 61,000 m³ (80,000 cu. yds.) in August, 1951.

The bottom valves were shut for the first time on the 14th March, 1952, a few months before the end of the dam completion. Later on the water storage will normally fill up from May to August and be used during the winter to increase the natural flow through the numerous downstream located power stations.

It has been necessary to move up hill the " Route Nationale de l'Iseran " (National Road number 202) ; the road was deviated, and 7 km. (4.4 miles) were reconstructed above the new lake, on the right bank ; 1,150 mètres (0.7 miles) of tunnels were drilled to give way to the new road. It was opened to the traffic during August, 1951. A junction road, 3 km. (1.9 miles) long, runs on the new dam crest and leads to the new Tignes village on the " Plateau des Boisses."

M. Vincent Auriol, Président de la République, formally inaugurated the Tignes Scheme on the 4th July, 1953.

Its main characteristics are summarized hereafter :

The Arch Dam

Height above the excavation-works bottom
180 mètres (590 ft.)

Height above the original river bed
160 mètres (525 ft.)

Crest altitude above sea level
1,792 mètres (5,880 ft.)

Foot of the dam thickness at the crown of the arch
43 mètres (142 ft.)

Thickness at the top 10 mètres (33 ft.)

Crest length, abutments included
400 mètres (1,320 ft.)

Concrete volume 630,000 m³ (825,000 cu. yd.)

The Reservoir

maximum level of the water 1,790 mètres (5,875 ft.)

lake area 270 hectares (667 acres)

useful capacity 230 millions m³ (186,000 acre ft.)

water intake tower flow
50 m³/s. (65 cu. yd./second)

water intake altitude 1,650 mètres (5,410 ft.)

total flow of the 2 emptying conduits
240 m³/s. (314 cu. yd./second)

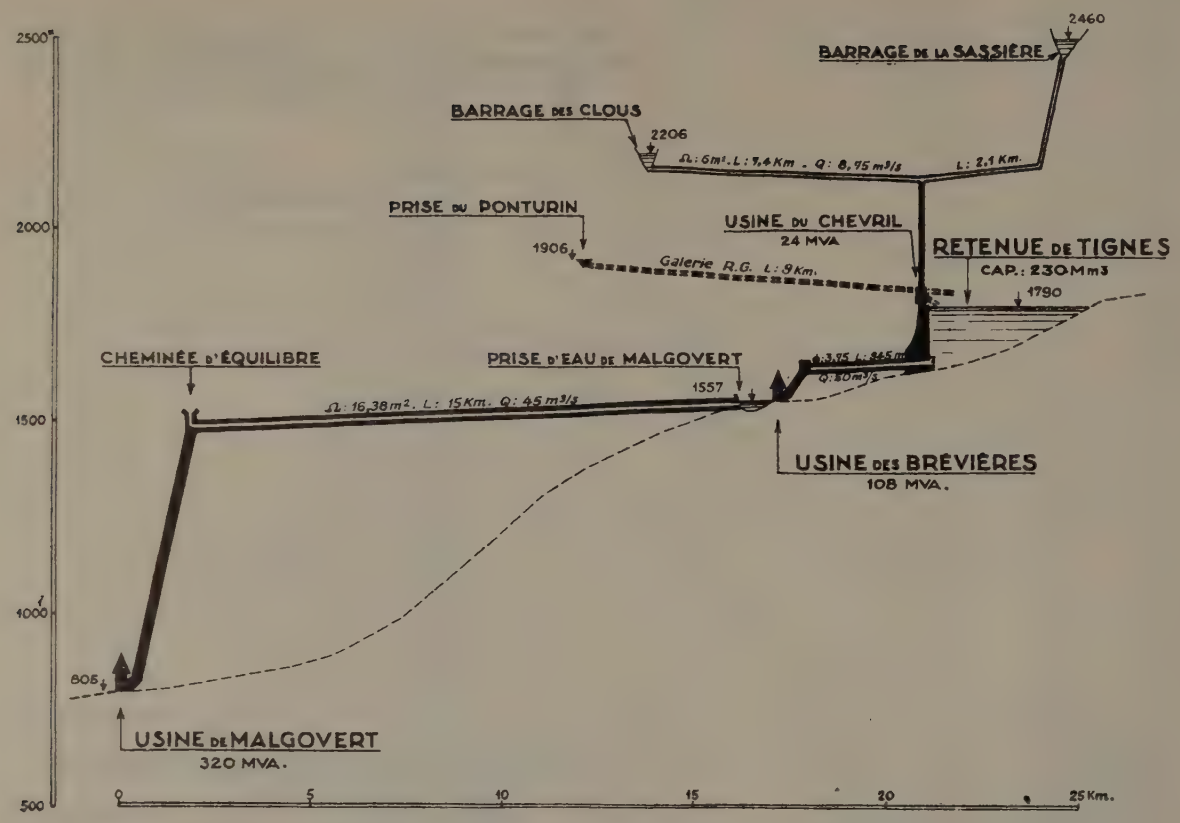


Fig. 5.—Workyard installations

The Brévières Power Station

The main object of the Tignes dam is the storage of a large volume of water at high altitude. But the dam provides too, a new head which is used at the Brévières power station.

The Tignes dam is connected to the Brévières plant through a gallery, 950 mètres (1,040 yds.) long, steel-lined and able to support the pressure of the reservoir when it is full.

The Brévières outlet canal altitude is 1,557 metres (5,100 ft.) and the maximum head of this plant is thus 233 mètres (765 ft.).

The station is harnessed with 3 vertical-axis Francis units running at 375 r.p.m. ; the total maximum flow is 50 m³/s. (65 cu. yd./second) and the total maximum output 130,000 hp.

The Malgovert Power Station

The Malgovert head race gallery.—The Malgovert water intake is located 1,200 mètres (0.75 miles) downstream of the Brévières outlet.

The Malgovert gallery runs into the slopes of the "Mont Pourri" mountain range, it is 15 km. (9.4 miles) long, has a cross section of 16 m³ (19 sq. yd.) and thus is one of the most important galleries in Europe. The most up-to-date devices were used to drill it, particularly in the last third of its length, where unusual difficulties occurred during the crossing of a water soaked sands section.

Special facilities of access were provided to lead to the various adits through which the drilling work was going on simultaneously. Thus the adit No. 9 was served by a cableway used for transport of both staff and materials ; adit No. 11 was easily reached by walking men and was served by a 500 kg. (1,100 lbs.) aerial ropeway for materials ; adit No. 13 was reached by jeep or half-track and was served by two cableways conveying the cement and the aggregates ; adit No. 14 was equipped with a telpher-line for men and materials ; adit No. 16 was accessible by road from Bourg St. Maurice.

From adit No. 1 to No. 12 (upstream side of the gallery) the drilling went on according to conventional methods. But around adit No. 13 traditional methods were impracticable ; there was to be crossed a stratum of triassic quartzite more or less finely ground and containing occluded water.

In May, 1947, the drilling of adit No. 13 was going on, when a serious incident stopped the work. At a depth of 72 mètres (79 yds.) the timber support was overburdened in a soil, made up of schist blocks wrapped in mud ; hollow noises were heard. On the 14th May at 8 a.m. it was decided to evacuate the main machinery, and at 10 a.m. the face wall broke ; the workers, running fast, escaped just in time from a flow of white liquid mud rolling huge schist blocks. The white mud looked like "tooth-paste" and was nicknamed after it by workers.

Adit No. 13 was abandoned and an adit No. 13 bis further downstream was started. But on the 5th April, 1949 the "tooth-paste" sprang again and broke the face wall ; barriers were quickly erected, only the fourth one was able to stop the flow.

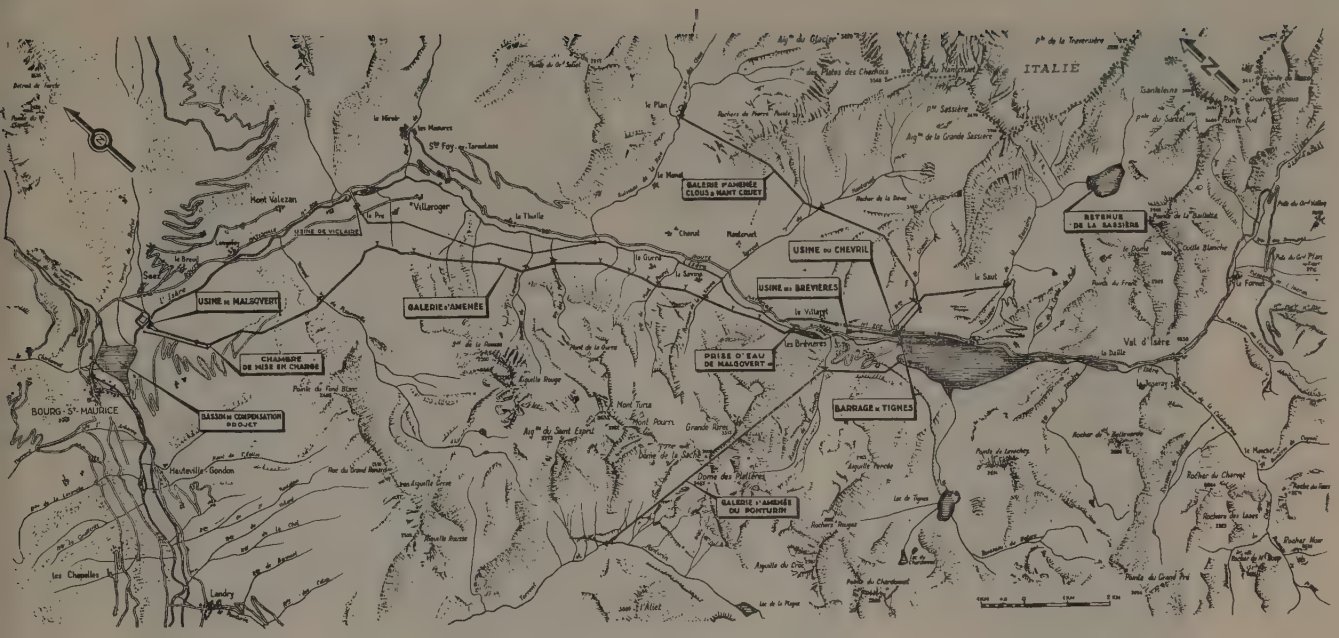


Fig 6.—Map of the Malgovert head race gallery

A thorough survey was then decided on, test borings were made to determine the geological seams and the thickness of the creeping sands. A new route was drawn for the gallery which had then to cross the water-soaked section over a distance of 150 mètres (165 yds.).

After studies and tests in the laboratory and on site, it was decided to dry and consolidate the moving ground by means of grouting with suitable materials : sodium silicate, colloïdal cement, and a petrifying solution of sodium silicate and phosphoric acid.

Along a length of 110 mètres (120 yds.) in highly treated grounds the gallery is lined as follows :

One full section steel support
One first safety concrete lining
One second reinforced concrete lining

We must leave now this difficult section of the gallery and come back to the normal tunnel.

The whole length of the tunnel was lined with concrete, mainly in order to support the soil ; a total concrete volume of 200,000 m³ (260,000 cu. yd.) was used for that purpose.

At the end of its 15 km. way through the " Mont Pourri " side, the gallery comes out to join the two penstocks at a point where is placed a huge non-spilling surge tank, 50 mètres (55 yds.) high, consisting of a cylinder, 18 mètres (20 yds.) inner diameter at the bottom, emerging at the top in a 30 mètres (33 yds.) diameter funnel ; the concrete lining of the surge tank is 1 mètre (1.1 yd.) thick.

The two penstocks are laid in a trench through the Malgovert forest, each one carries a flow of 25 m³/s. (33 cu. yd. per second) under a gross head of 745 mètres (2,450 ft.) making a power available of 216,000 hp. each. This output is a penstock world record. The upper parts of the penstocks are made of 2,20 m (7.20 ft.) diameter pipes, banded with high resistance steel wire slings ; the lower end pipes are hooped with rolled steel rings.

The Malgovert Power Station.—The power house is a rectangular building located on the left bank of the river Isère, at 2 km. (1.25 miles) east from Bourg St. Maurice. The engine room is equipped with two gantry cranes, 100 tons each.

Four 108,000 hp. generating units are made up of one alternator placed between two single-runner double-jet Pelton turbines. The turbines are among the most powerful Pelton type ever made.

The maximum flow of each injector, 2,75 m³/s. (3.6 cu. yd. per second) gives a maximum flow of 5,5 m³/s. (7.2 cu. yd. per second) per runner and a 44 m³/s. (56.6 cu. yd. per second) total flow for the power station.

Each injector has a deflector for a quick jet diversion ; the diversion is followed by a slow regulation obtained by moving the injector needle at small speed in order to avoid any important over pressure in the penstock. One of the two runners of each generator set is equipped with a braking-jet.

The rated data of the alternators are as follows :

—output	80 MVA
—power factor *cos.* φ	0.88
—voltage	10.3 kV.
—current	4750 A.
—speed	428 r.p.m.
—overspeed	770 r.p.m.
—Flywheel effect	PD^2 — 900 t.m.² WR^2 — 5,340 lb. sq. ft.

Each alternator weighs 336 tons.

In fact, the alternators have been able to deliver a 80 MN active output ; moreover they have been designed to consume a 67 MVA reactive power, *cos.* φ =0, with a field current yet positive, in order to set alive the intended 380 kV lines.

The alternators' manufacturer was free to select the stator voltage, and the rotor flywheel effect ; indeed, the alternators' natural WR^2 fitted well to the turbines ; the manufacturer was then able to design generators with a very high rate in the use of materials, and to place them well above any other powerful and up-to-date alternators with salient poles.

It is interesting to note that the " Pragnères " power station in the Pyrénées, will be equipped with

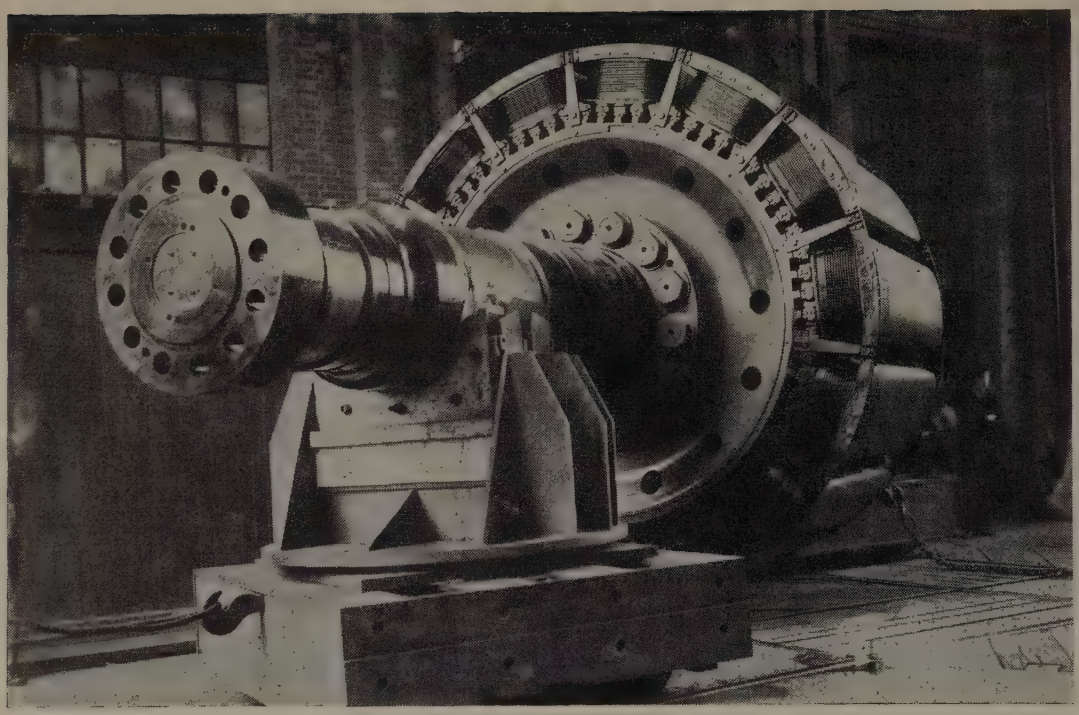


Fig. 7.—Generator rotor

alternators identical to the Malgovert ones. The "Pragnères" Pelton turbines will somewhat differ because of the head which is higher at Pragnères: 1,250 mètres (4,100 ft.) instead of 745 mètres (2,440 ft.).

Each Malgovert alternator delivers its energy to the network through a 80 MVA transformer 10.3/220 kV.

The outdoor station has 220 kV and 150 kV bays; it will be completed later with 380 kV bays. The power is despatched to the French network, and through a 220 kV line to Italy (Aosta).

SECOND PART
—Some of the Electro-Mechanic Problems Involved

As already seen, the Malgovert generating sets are the most powerful sets built up to now with a horizontal shaft running at slow speed. This second part is devoted to some of the new problems which had to be solved to design, build, transport and erect such large generators. I cannot, indeed, discuss all of them, and shall report briefly three questions related to:

—the generator rotor construction
—the bearings construction
—the generator transports from the factory workshops at Belfort to site at Malgovert.

The Rotor Construction

The shaft of the generator rotor is about 10 mètres (33 ft.) long, the rotor diameter is 3 mètres (10 ft.) without poles, and the pole turning diameter, 3.55 mètres (12 ft.).

The conventional method of forging and machining a one piece rotor of such dimensions was out of the question, the ingot weight would have been 300 tons, and the chances of rejection too great. The hot stacking of steel discs on a single shaft was set aside too, for fear of excessive stresses.

It was decided to stack and press together non-borel steel discs, by means of alloyed steel rods, between the end plates of two shafts.

Corresponding to the 14 poles and 14 rotor planes of symmetry 14 tie rods were hot bolted through the discs in order to obtain the necessary stress after cooling. The factor of safety is such that one out of two rods could break without danger to the good operation of the rotor.

The discs being without central holes, it was possible

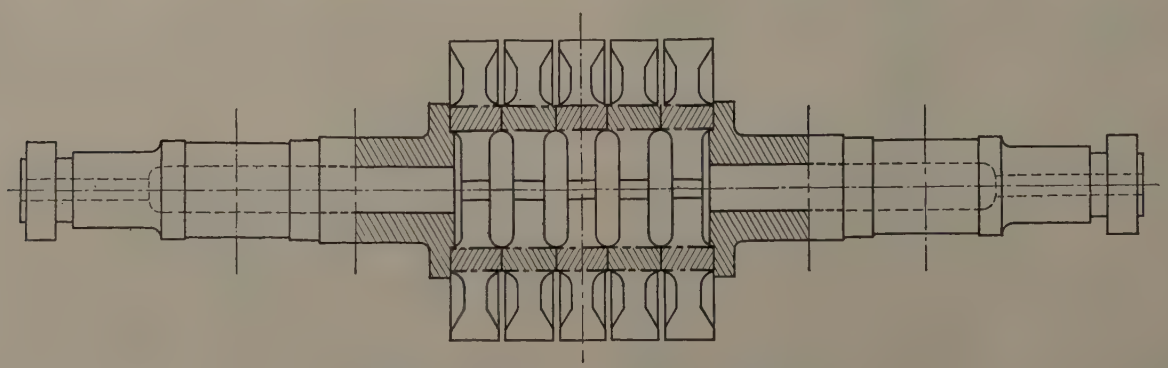


Fig. 8.—Generator rotor cross section

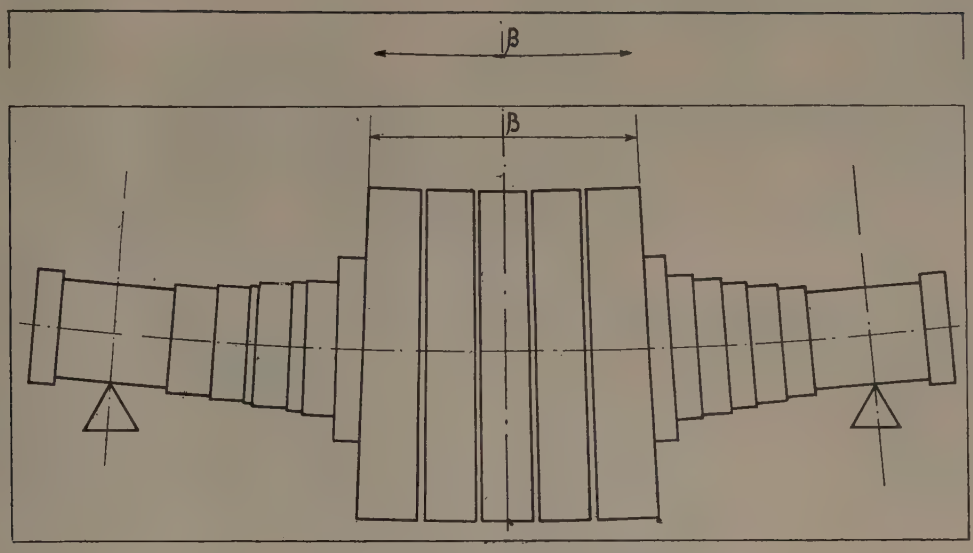


Fig. 9.—Rotor model bending

to thin them at the centre, leaving with full thickness only an exterior ring, and thus to lighten the rotor by 40 tons, a factor which helped to solve other numerous problems.

This new type of construction left unknown the rotor rigidity and its critical speed ; they were first calculated by comparison with a steel tube having a thickness corresponding to that of the annular part of the discs ; but this was not sufficient and two kinds of tests were carried out with reduced models.

The first model was made up of stacked discs designed in accordance with the laws of similitude, and was loaded under an off centre stress.

The sag was determined by the measurement of the angle taken by the extreme faces of the disc stack, and the moment of inertia was found 1.4 times larger than the one given by mere calculations.

The second test device, another reduced model with an adjustable elasticity support, shown on the next slide, was submitted to alternative flexion stresses of variable frequency and the vibrations' elongation was measured with a cathode-ray oscilloscope.

Both kind of tests gave the same critical speed : 1,230 r.p.m. ; this being above twice the rated speed (2 × 428 r.p.m.) and well above the runaway speed (770 r.p.m.), was suitable.

After these elaborate tests it was quite safe to build the generator rotor according to the new design.

The Bearings

The Malgovert bearings are among the most important—if not the most important—ever built up to now for horizontal shaft generating sets.

The whole revolving part of the generating set weighs 204 tons and is supported by two bearings ; the weight divides equally between the bearings and is increased by the thrust of the turbine jets ; the largest stress upon one bearing occurs during operation at half load, with one turbine running at its full load, the other turbine remaining off load ; the maximum stress on the corresponding bearing is then 135 tons.

The question was raised of adopting the same technique that was used for the 50,000 kVA set of Portillon power plant, where the bearings were lubricated by means of oil-rings and cooled by circulation of water through the bearing bushes ; that technique was worth noticing, it had proved simple, efficient and safe at Portillon. But a large step remained to be overcome to pass to the Malgovert characteristics :

—maximum load, 135 tons against 80 tons

—turbines overhang 1.90 m (6. 2/10 ft.) against 1,35 —mètres (4. 1/2 ft.)

—etc. . .

An overheating of the film of oil and excess of leakage through the ends of the bearing bush were to be feared.

These points were thoroughly investigated both by calculations and model tests.

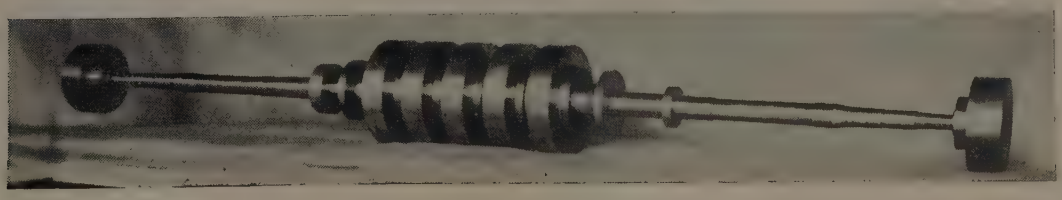
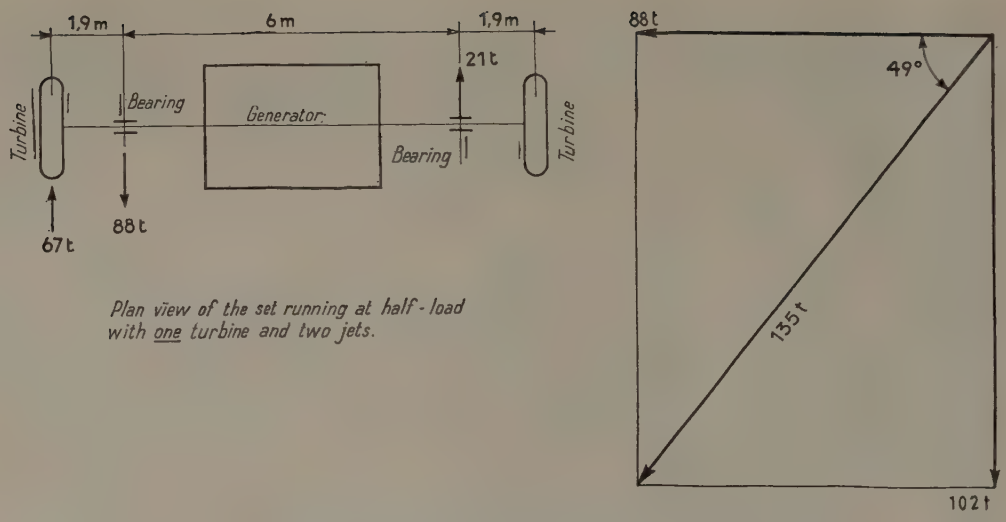


Fig. 10.—Second rotor model



Plan view of the set running at half-load with one turbine and two jets.

Fig. 11.—Stresses on one bearing

The Lehmann method of study to determine the heat flow showed that the temperature of the oil film could be kept between reasonable limits by means of a judicious arrangement of the water piping inside the bush babbit metal.

The main items of information collected through tests upon the model are grouped on the next slide, where are represented the oil-pressures variations around and along the journal. During the model tests the bush temperature and the oil temperature were varied systematically through a variation of the water flow; and the best operation point, giving together low friction losses and a very high factor of safety, was found to be around a 60°C. temperature of oil and a cooling water temperature rise of 12 to 15°C.

The test results were so satisfactory that it was decided to give up the previous plan for lubrication with circulation of oil through coolers placed outside, and its inherent complications.

Thus, the Malgovert bearings are, like the Portillon ones, plain bearings with automatic oiling rings, and cooling by circulation of water inside the bush. However, an oil pump has been added to spread oil plentifully on the journal before and during the starting up; the oil pump may be kept running during normal operation of the generating set and give an over-oiling, but that is not necessary, the oil rings are sufficient for a normal lubrication; and it is possible to replace or repair an oil-pump without shutting down the generating set.

The Generator Transports

The problems involved by the transport of the generators from Belfort to Malgovert by road were difficult. The stator weighs 105 tons and has been transported upon a special trailer made of two large girders, more than 16 m (52 ft.) long, laid forward and backward upon cross-bars set themselves upon four half-axle boggies. The convoy was hauled by two "Pacific" tractors, 300 hp. each; its total weight

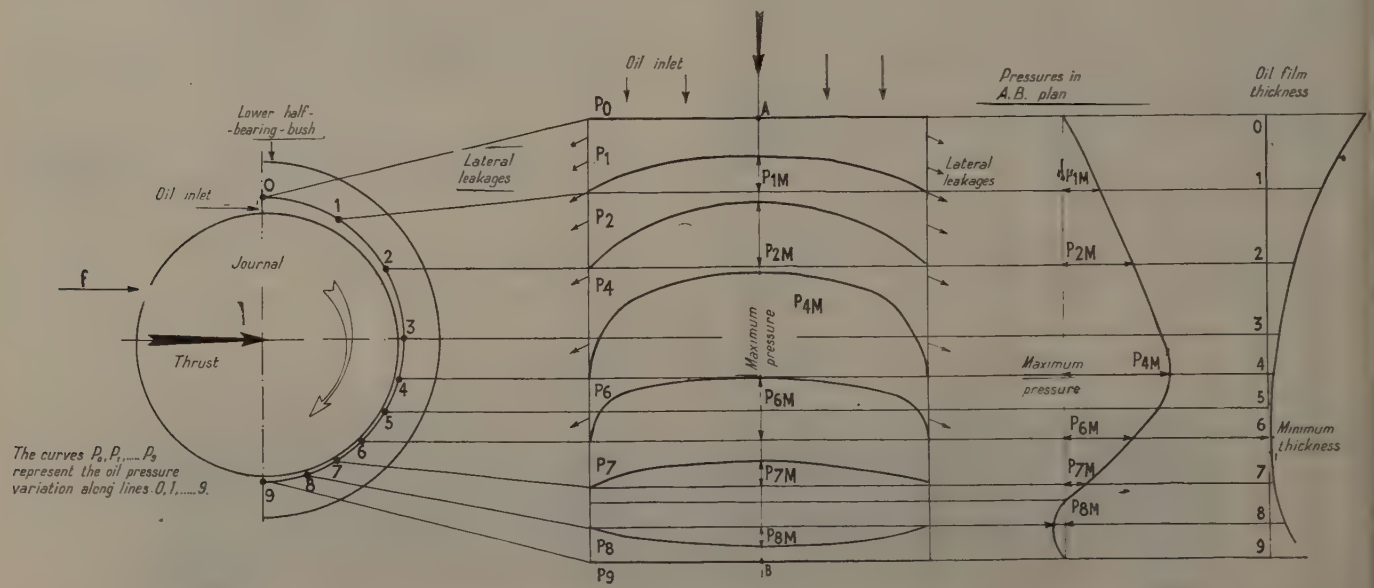


Fig. 12.—Oil pressure variations around and along the journal

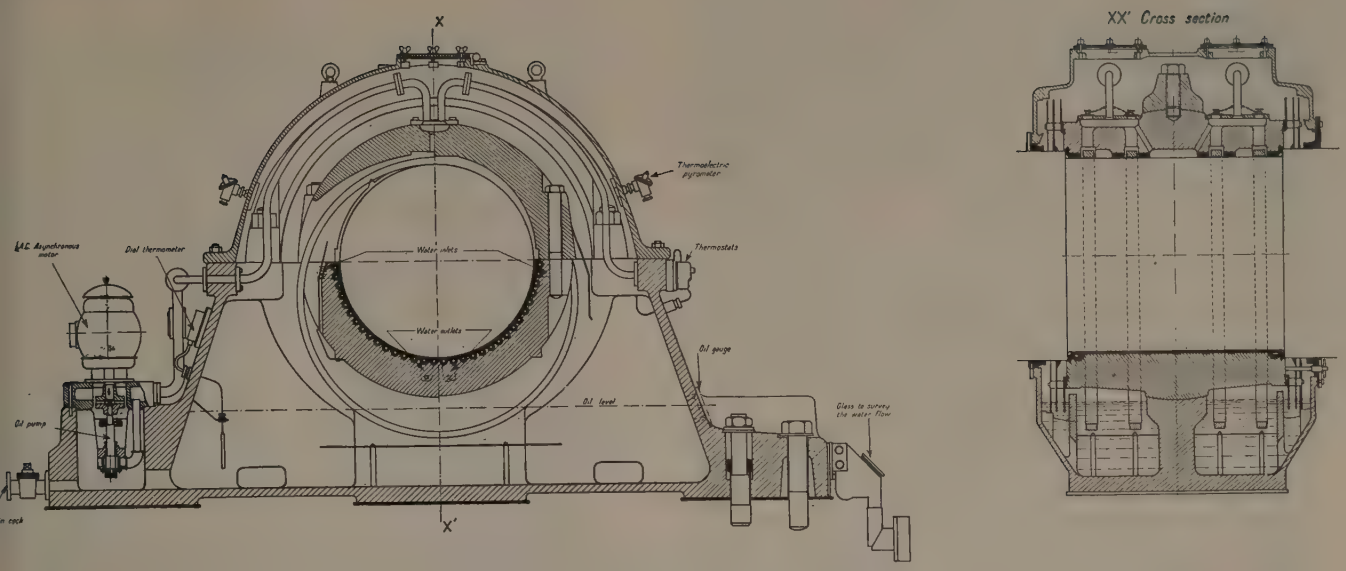


Fig. 13.—Cross section of one bearing

was 180 tons, its height above ground, 5,6 mètres (18 ft.). Its passage under a railway line at Notre Dame de Briançon obliged to raise by 1,30 mètres (4 ft.) the railway bridge structure ; this work was made four times—one for each of the four stators—at night and without trouble for the railway traffic, and was repeated when the Malgovert transformers passed at the same point. The slide shows the bridge raised by jacks, and lifting with itself a part of the railway track.

The rotor transport problems were different ; the total cross section was less, but the weight of each rotor was 170 tons, and its length reached 10 mètres (33 ft.). Each rotor was loaded upon a special trailer with 48 solid rubber tyre wheels. The total weight is divided between 12 axles and allow the passage upon bridges forbidden to the stator convoy whose axles were more heavily loaded ; the way followed by the rotors was thus slightly shorter. The rotor convoy was hauled by two " Pacific " tractors, and one 200 hp. " Diamond " tractor.

The two 100 ton gantry cranes of the Malgovert engine room unloaded the four stators and four rotors.

Conclusions

I am afraid I have only been able to evoke too briefly a few of the numerous difficulties which have been solved by technicians of all kinds—geologists, hydrologists, civil engineers, contractors, manufacturers,

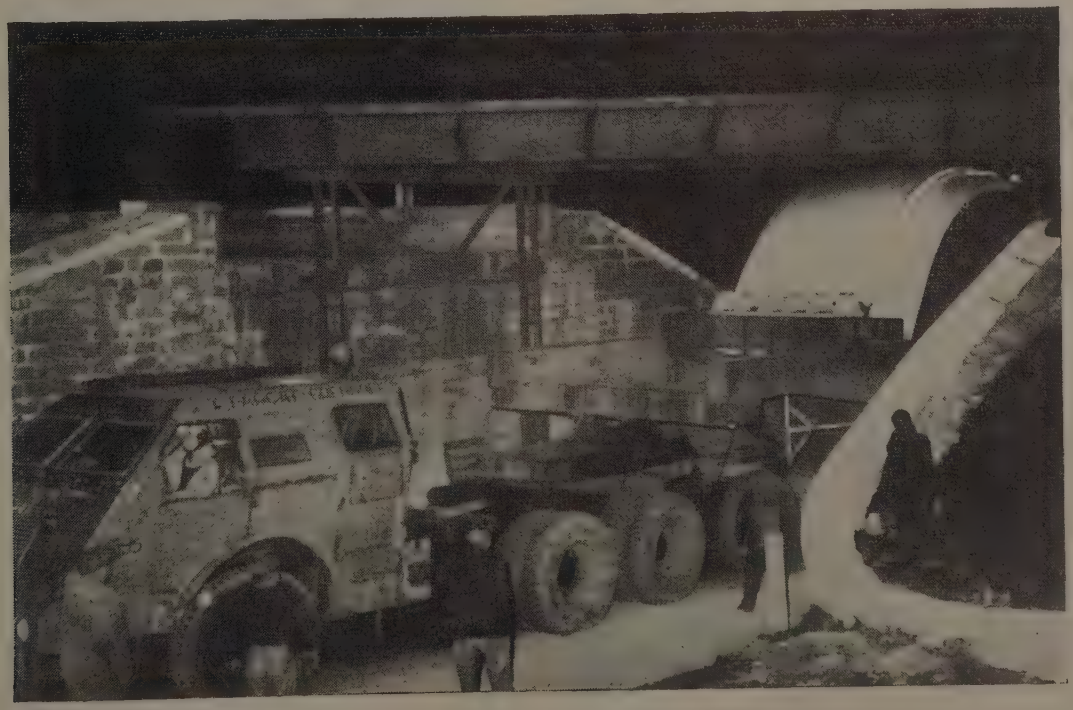


Fig. 14.—The stator passing by night under a lifted railway bridge

electricians—to achieve this vast scheme, which, under the general name of " Tignes," include the highest dam in Europe, two main power stations at Brévières and Malgovert, and various important works such as the remarkable Malgovert galleries and penstocks. All of them are responsible for the achievement and success of this project, new example of the vast possibilities of co-operation and team work.

If, being necessarily too short, I have deceived you, I hope anyway that I have given you the desire to learn more, to see more, and to come over to France to visit our installations ; it will be a great pleasure indeed to see you again and to show you over those of our works and plants you are more interested in.

Acknowledgments

Maitre de L'oeuvre (Direction de l'Equipement de l'Electricité de France :

Directeur de l'Equipement	M. R. Giguet
Contrôleur général pour les Alpes	M. Thaller
Directeur de la Région Alpes I	M. A. Ligouzat
Chef du Service Etudes	M. F. Combe
Chef d'arrondissement de la Hte Isère	M. J. Audy
Chef d'aménagement des Brévières	M. J. P. Payan
Chef d'aménagement de Malgovert	M. J. H. Rousselle

Etude du Barrage de Tignes (Tignes dam design) :
Messrs Coyne et Bellier

Architectes :

Barrage de Tignes	Messrs Labro et Gleize
Centrale des Brévières	M. Verrier
Centrale de Malgovert	M. Chomel

Entrepreneurs de Genie Civil (Civil engineering contractors) :

Entreprise	Industrielle	Entreprise	Anselmo
,,	Billiard	,,	Bachy
,,	Sotrabas	,,	Arnaud-Galaup
,,	Rossetti	,,	Travaux Souterrains
,,	Dalver	,,	Soletanche

Construction du Materiel (Machinery makers) :

Conduites forcées (penstocks) de Malgovert	Et. Bouchayer et Viallet
Vannes du barrage (dam gate valves)	Et. Neyrpic
Turbines de Malgovert	Et.Neyrpic et At.du Creusot
Conduites, turbines, ponts roulants des Brévières	Ateliers du Creusot
Alternateurs des Brévières	Le Matériel Electrique S.W.
Alternateurs de Malgovert	Société Alsthom
Ponts roulants (gantry cranes) de Malgovert	Applevage et Soretex
Transformateurs des Brévières	Cie Electro-Mécanique
Transformateurs de Malgovert	Sté Savoisienne
Disjoncteurs (circuit breakers) de Malgovert	At. de Delle et Sté Galileo

Main d'Oeuvre (Labour :)
Twelve million days of work from staff of every rank (workmen, employees, foremen, engineers and management).

Correspondence

The Institution, whilst being at all times pleased to open its columns to correspondence, cannot accept responsibility for the opinions expressed.

To the Editor of THE STRUCTURAL ENGINEER

Draft Revision to B.S. 449

Sir—May I refer to Mr. L. Kraus' remarks concerning the Draft Revision of B.S. 449? It is Mr. Kraus' opinion that the suggested design rule for single angle tension members is more complicated than its predecessors. This I cannot appreciate after a re-reading of para. 39 (b) pf. B.S. 449 (1948).

A careful search failed to reveal any quantitative reasoning on which the old rules are based ; they seem to rest on no foundation other than intuition and long usage.

Publication No. 7 of the British Constructional Steelwork Association shows clearly that application of these rules to all angle sections results in a variable factor of safety. It also shows the achievement of more uniform factors using the proposed rule. This results in marked economy and though the proposed rule, as stated, is empirical, in a more rational approach to the design of such members.

Yours, etc.,
H. M. NELSON,
Assistant Professor of Civil Engineering,
Royal Military College of Canada.

To the Editor of THE STRUCTURAL ENGINEER

Sir—Having read Professor H. M. Nelson's comments I am still unrepentant. I still consider the new rule for single angle tension members to be more complicated than the present B.S. 449 (1948) rule.

In order to work to the new rule (without going through the working of the formula each time) it will be necessary to have revised tables of safe tension tabulated for each single angle member attached first by one leg and then by the other (if an unequal angle).

It seems that we may as well test each and every case experimentally, divide by the same factor of safety throughout and tabulate these values without bothering about any academic argument as to how much of the outstanding leg is effective. This procedure would at least be straightforward and give a uniform safety factor.

I have now read Professor Nelson's report in B.C.S.A. publication No. 7 and sympathise with his feelings regarding my criticism after all his efforts, but would point out that his largest percentage gains taking rule (111) as a basis are for angles connected by the short leg. In my experience this type of connection is usually adopted because the tension member is subject to reversal and this reversal condition is the criterion for design, hence the apparent percentage gain will never be realised.

Finally I can hardly consider the end connections of the test members to be typical of the details used in practice, especially with regard to the 25/32 in. dia. bolts in $\frac{3}{4}$ in. dia. holes, and I imagine that this might give higher allowable loads than in the case of members with $\frac{1}{16}$ in. tolerance holes.

Yours, etc.,
L. KRAUS.

To the Editor of THE STRUCTURAL ENGINEER

The Ultimate Strength of Prestressed Concrete Beams

Sir—Referring to the paper by Professor H. J. Cowan in the July issue of THE STRUCTURAL ENGINEER, I should like to make the following observations.

The statement "that beams should have a load factor of at least 2 against primary tension failure, which depends essentially on the ultimate strength of the steel": does not this depend upon the amount of reinforcement in primary tensile failures? It requires a very small percentage of steel to fail by fracture of the steel at the ultimate; most of these types of failure occur when the strain is between the yield value and the ultimate, when the neutral axis rises rapidly and the concrete crushes.

It seems a curious anomaly that in ordinary reinforced concrete beams the ultimate concrete stress is found to be around 0.8 of the cylinder stress, whereas in prestressed beams the ultimate concrete stress is found to be equal to or greater than the cylinder stress. This is presumably because the neutral axis is found to be so high at failure.

The value of the strain factor (designated K' by Professor H. J. Cowan and F' by Professor A. L. L. Baker, who first introduced it) for non-bonded cables depends chiefly upon the disposition of the loads on the beam and the length of the cable between anchorages. It is also influenced to some extent by the amount of prestress.

A value of 1 for K' should be used for all cables which have been properly grouted or in beams where the concrete is cast around the pretensioned wires; deformed high tensile bars and wires, now being manufactured, and expanding grout should do much to ensure a proper bond with the concrete.

It appears to me that there is something fundamentally at fault with all the formulae yet produced for the calculation of pre-stressed beams at failure.

For instance, if, in an over-reinforced prestressed beam at failure by concrete crushing, the load were removed, the beam will return almost to its original state, showing that the prestressing force is still active at failure; this effect is not considered or allowed for in the formulae for failing moment.

I consider that it should be postulated that the prestressing *moment* remains constant throughout the life and length of the beam.

The prestressing force is reduced, however, since if the above postulate were true, the prestress remaining in the cable at the critical section will be $f_{sp}\frac{e}{a}$, where e is the original eccentricity at prestressing and a is the final lever arm after cracking.

If the above is true, then the ultimate compressive force in the concrete can be increased by $A_s f_{sp}\frac{e}{a}$, when computing primary compressive failures, since the prestressing moment is of opposite sign: then there will be no need to use or assume such a high strength for the beam concrete.

Similarly, the amount of stress available in the steel to resist the bending moment before reaching its yield value will be $f_y - f_{sp}\frac{e}{a}$, which is a greater margin than usually taken.

The secant value of E'_s will also be higher at the yield value.

Yours, etc.,
A. J. ASHDOWN,
(Associate-Member).

Professor Cowan replies:—

The proposed load factors for primary tension and compression failures are those given in the Institution's First Report on Prestressed Concrete, to which reference is made in the Introduction to the paper. The Author agrees with Mr. Ashdown that a beam with a very small percentage of steel would fail by fracture of the wires, and a higher load factor is then desirable. It is, however, better to avoid this type of failure altogether in the design, and the First Report accordingly recommends a minimum percentage of prestressing steel. As shown in the paper, this is obtained from the limiting neutral axis depth ratio for steel rupture, n_r/d, of equation (9), and the minimum percentage stipulated in the First Report appears to provide an adequate safety margin against steel rupture.

The Author's method does not require a different ultimate concrete stress for normal reinforced concrete and for prestressed concrete. Using the equivalent rectangular stress block employed by Prof. A. L. L. Baker, however, it would be necessary to make an adjustment of the kind described by Mr. Ashdown due to the low depth of the neutral axis at failure in prestressed concrete beams showing primary tension failure.

Mr. Ashdown's comments on the strain factor are valuable. It is evidently desirable to ensure that the strain factor is near to unity in all prestressed concrete beams, to obviate a premature concrete failure.

Although Mr. Ashdown is correct in stating that the prestressing force is still active in a concrete beam which has failed through crushing of the concrete, he would surely concede that the beam is no longer capable of carrying the load. The beam has therefore failed, and the equations for its ultimate load carrying capacity must be computed accordingly.

The Author has some difficulty in following the last paragraphs of Mr. Ashdown's contribution. The secant value of the modulus of elasticity of the steel is a physical property of the steel, and it *decreases* as the steel stress increases. The ultimate strength of the concrete, as defined by the Author, is also a physical property of the material. These quantities cannot be varied at will. Mr. Ashdown presumably does not refer to the loss of prestress which occurs over a period of time due to creep etc. When the beam is loaded, the prestressing force is bound to increase due to the extension of the eccentrically placed steel as the beam deforms. In the case of a primary tension failure, this eventually results in the yielding of the steel and the failure of the beam. In the case of a primary compression failure the steel does not yield, but there is no reduction in the magnitude of the prestress; this is easily confirmed by experiment.

Institution Notices and Proceedings

IMPORTANT NOTICE TO SUBSCRIBERS

Subscribers, Newsagents and Subscription Agencies at home and abroad are asked to note that as from the 1st January, 1955, copies of THE STRUCTURAL ENGINEER can only be obtained on application to the Secretary of the Institution of Structural Engineers, 11, Upper Belgrave Street, London, S.W.1.

Subscription Rate

The Annual Subscription, post free, to any part of the world is 44s. 6d.

Single Copies

The price of single copies is 3s. 6d. or 3s. 9d. if sent by post.

In order to fulfil all demands, subscribers and others are requested to state their requirements as soon as possible.

The usual rate of discount will apply.

VACANCY FOR DEPUTY ASSISTANT SECRETARY OF THE INSTITUTION

The Institution of Structural Engineers invite applications for the office of Deputy Assistant Secretary. The applicants must be qualified engineers with executive and administrative experience, between 30 and 50 years of age. Commencing salary will be not less than £1,100 per annum. The appointment will be in the first instance for a probationary period of one year and subject to satisfactory service the successful applicant will be promoted to Assistant Secretary of the Institution in September 1959 with prospects of appointment to the Secretaryship of the Institution in due course. Pension fund in operation, 5 per cent. on either side. Applications giving full particulars of education, career and experience to the Secretary of the Institution, 11 Upper Belgrave Street, London, S.W.1, marked in top left hand corner " Deputy Assistant Secretary," should be received by 31st January, 1956.

GENERAL MEETING

A General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, October 6th, 1955, at 6 p.m., when the Presidential Address for the Session 1955-6 was given by Mr. Stanley Vaughan, B.Sc., M.I.C.E., M.I.Struct.E., A.C.G.I. The Retiring President, Dr. S. B. Hamilton, O.B.E., M.Sc., Ph.D., B.Sc.(Eng.), A.R.C.S., M.I.C.E., M.I.Struct.E., was in the Chair.,

The Chairman welcomed the guests who were present and presented the following awards :

Bronze Sessional Medals for the best paper read before the Institution during the Session 1953-4, to Mr. J. S. Terrington and Dr. J. M. Hawkes for their paper on " Crane Gantry Girders for Steelworks."

Bronze Research Medals to Dr. R. H. Wood for a paper on " A Derivation of Maximum Stanchion Moments in Multi-storey Frames by Means of Nomograms " ; and to Dr. P. B. Morice and Mr. G. Little for a paper on " Load Distribution in Prestressed Concrete Bridge Systems."

The Chairman then paid a tribute to Mr. Vaughan's work for the Institution and invested him with the Presidential Badge. Mr. Vaughan, who then took the Chair, expressed his thanks and called upon Mr. E. Granter (Past President) to propose, and Mr. L. Scott White, O.B.E. (Past President) to second a vote of thanks to Dr. Hamilton for his work as President of the Institution during the Session 1954-5.

The vote of thanks was carried and Dr. Hamilton responded briefly.

Mr. Vaughan then gave the Presidential Address for the Session 1955-6, which is printed in this issue.

At the conclusion of the Address, a vote of thanks to the President was proposed by Lt.-Col. G. W. Kirkland, M.B.E. (Hon. Editor), and seconded by Mr. F. M. Bowen (Member of Council).

The vote of thanks was carried with acclamation and the President briefly expressed his appreciation. The proceedings then terminated.

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 27th October, 1955, at 5.55 p.m., Mr. Stanley Vaughan, B.Sc., M.I.C.E., M.I.Struct.E., A.C.G.I. (President) in the Chair.

The Minutes of the Ordinary General Meetings held on the 26th May and 23rd June, 1955, as published in the Journal, were taken as read, were confirmed and signed.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections as tabulated below should be referred to when consulting the Year Book for evidence of membership.

GRADUATES

BASON, John Roland Thomas, of Surbiton, Surrey.
BROOKE, Jack, of St. Helens, Lancs.
COLEMAN, George Martin, of Cape Town, South Africa.
FEARNLEY, James, of Turton, Nr. Bolton, Lancashire.
GANDHI, Vashu Jiaram, of Bombay, India.
GOODCHILD, George, of East Barnet, Herts.
HERMAN, Terence Francis, of East London, South Africa.
HUTCHINSON, David Alan, of West Hartlepool, Co. Durham.
JONES, Clifford Ivor, of Barking, Essex.
LAU YAU CHUN, of Hong Kong.
LEVI, Albert, of London.
MARTIN, Derek Kenneth, of Pinner, Middlesex.
MOORE, John Neville, of London.
MUSTAPHA, Bin Hussain, of Penang, Malaya.
OBORNE, David John, of London.
ROBERTS, Joseph Frank Robert, of Johannesburg, South Africa.
SALMINS, Artis Ilgonis Robert, of Bradford, Yorkshire.
TSANG YIN SANG, of Hong Kong.
WARD, Denis Robert, of Norwich, Norfolk.

Associate-Members

Adams, Hugh Morley, B.Sc.(Eng.) Natal, of Durban, Natal, South Africa.
Allan, Benon Jakub, P.Eng., A.M.I.C.E., of Lenzie, Scotland.
Barton, John Richard, of Guildford, Surrey.
Beck, John Stanley, of Stockport.
Burgess, John Holland, Hale, Nr. Altrincham, Cheshire.
Capps, Derrick Ernest, of Hillingdon, Middlesex.
Cheng Chung Chow, B.Sc.(Civil) Leeds, of Kowloon, Hong Kong.
Craddock, Harry Norman, of London.
Crawford, James Henry, of Mombasa, Kenya.
Downes, Henry George Edward, of London.
Durkin, Kenneth Ronald, B.Sc.(Eng.) London, of Bognor Regis, Sussex.
Ehsanullah, Ehsan, of London.
Ellis, Tony James, of Southall, Middlesex.
Farnborough, Robert Furnberg, B.Sc., of London.
Fowke, Bertram William, A.M.I.C.E., of London.
Gibbs, Douglas Rutherford, B.Sc.(Eng.) London, of Umtali, Southern Rhodesia.
Goddard, Philip Dennis, of Wembley, Mfddlesex.
Hadjicosta, Costa George, of Cyprus.
Hardiman, Noel James, B.Sc.(Eng.) London, of Bexleyheath, Kent.
Higgs, Thomas Fallon, of Liverpool.
Jha, Krishna Mohan, of Muzaffarpur, Bihar, India.
Kirk, Sidney Edward, of London.
Kotnis, Vishvanath Ramchandra, B.E.(Civil) Bombay, of Poona, India.
Kung Shien-Cho, of Hong Kong.
Lethbridge, John Anthony, B.Sc.(Eng.) London, of Brookmans Park, Herts.
Lewis, Wyndham Stuart, B.Sc.(Civil) Birmingham, of London.
Minney, Arthur Joseph, of Kettering, Northants.
Moses, Charles, B.Sc. Bombay, of Fort, Bombay, India.
Neave, Harry, of Merstham, Surrey.
Pinkney, John Dennis, Ashby, Scunthorpe, Lincs.
Pokorny, Milos Karel, of Johannesburg, South Africa.
Robertson, Alexander Stuart, B.Sc.(Civil) Glasgow, of Kowloon, Hong Kong.
Simpson, John Edward Henry, B.E.(Civil) Liverpool, of Liverpool.
Sneath, Norman, M.A.(Cantab.), of Canberra, A.C.T., Australia.
Thair, Arthur Ernest, B.Sc.(Eng.) London, of Northwich, Cheshire.
Tsang Foh Kwei, of Singapore.
Wells, William John, A.M.I.Mun.E., of Hamilton, Lanarkshire, Scotland.
Wojakowski, Andrew Walter Stanley, B.Sc.(Eng.) London, of London.

Transfers

Students to Graduates

Allen, Derek, of Harrow, Middlesex.
Atkinson, John Stuart, of Ashby, Scunthorpe, Lincs.
Bradnick, Robin James, of Belfast.
Bull, Peter John, of London.
Crossley, Norman, of Manchester.
Fawcett, Frank Birkinshaw, of Ealand, Nr. Scunthorpe.
Forrest, Esli James, of Papatoetoe, Auckland, New Zealand.
Gandhi, Kaikobad Manecksha, of Bombay, India.
Gentry, Leslie John, of Johannesburg, South Africa.
Goodey, Donald Alfred Thomas, of Ndola, Northern Rhodesia.
Howes, Roy, of Bolton, Lancs.
Jackson, Norman, of Sheffield, Yorks.
Jones, Barry Alfred Champion, of London.
Jones, Brian Vernon, of Christchurch, New Zealand.
Lambert, Michael Francis, of Bolton, Lancs.
Lefever, Geoffrey James, of London.
Levick, Sidney Isaac, of London.
Mehta, Navinchandra Raghunath, of London.
Moore, Ronald Ernest, of Belfast.
Morgan, John Wynford, of Wigston, Leicester.
Nicholas, Wilfred Arasaratnam, of London.
O'Grady, Carl Robert, of Auckland, New Zealand.
Parrott, John Frank, of Scunthorpe, Lincs.
Ratcliffe, Keith Urwin, of Belper, Derbyshire.
Singh, Matharu Mohan, of Nairobi, Kenya.
Slack, George Edwin, of Scunthorpe, Lincs.
Smith, James Frank Hinton, of Bristol.
Stock, Stanley Gordon, of Bristol.
Thomas, Martyn Robin Haden, of Birmingham.
Thurston, Clifford Charles, of Johannesburg, South Africa.
Tune, Donald Hector, of Scunthorpe, Lincs.

Student to Associate-Member

Herring, Bernard Carroll, of Salisbury, S. Rhodesia.

Graduates to Associate-Members

Allott, Ernest James, B.Sc.(Eng.) London, of Rotherham, Yorks.
Archibald, Allan Edwin, B.Sc.(Eng.) Rand, of Johannesburg, South Africa.
Bacon, Roger, B.Sc.(Eng.) London, of Epsom, Surrey.
Baikoff, Eugene Marc Alexander, of Bergvlei-Tvl, South Africa.
Bailey, Sidney, of London.
Banerjee, Butto Krishna, B.E., B.Sc. Calcutta, of Stafford.
Carling, Peter George, B.Sc.(Civil) Wales, A.M.I.C.E., of Swansea, Glam.
Carter, Clifford, of Salford, Lancs.
Cartwright, John Desmond, of Birmingham.
Case, John Kenneth, of London.
Case, Lionel Francis, of London.
Clarke, Stanley, of Garston, Liverpool.
Cohen, Maurice, B.Sc.(Eng.) London, London.
Davies, Colin, B.Sc.(Eng.) London, Treorchy, Rhondda, Glam.
Davis, John, of London.
Douglas, Roderick Emmanuel, of Salford, Lancs.
Drummond, Brian Godfrey, B.A.(Cantab.), of Hassocks, Sussex.
Dunthorne, Gareth John, of London.
Du Toit, Pieter Schalk, of Johannesburg, South Africa.
Feit, Michael John, of Ashford, Middlesex.
Folland, Alfred John, of Port Talbot, Glam.
Foxwell, John, of Leigh, Lancs.
Fraser, Angus Stuart, B.Sc.(Civil) Natal, of Bulawayo, S. Rhodesia.
Gardner, Roger Whiteside, B.Sc.(Eng.) London, D.I.C., A.M.I.C.E., Barnsley, Yorks.
Gee, Jeffrey Ernest Malcolm, of New Barnet, Herts.
Gibson, John Hartt, B.Sc.(Eng.) London, of Sutton Coldfield, Warwicks.
Grahame, Russell Godfrey, of Accra, Gold Coast, Africa.
Graney, Dennis Francis, of Hounslow, Middlesex.
Henderson, John Graham, B.Sc.(Civil) Durham, of Billingham, Co. Durham.

HIXSON, Robert Ivor, of Swansea.
HOBIN, Richard Edward, of London.
HORWELL, David Gordon, of Twickenham, Middlesex.
HUSAIN, Mahmood, of London.
HUSCROFT, Leslie, of Rotherham, Yorks.
JAMES, Bryan Wilson, of Porirua, Wellington, New Zealand.
LANGDON, John, B.Sc.(Tech.) Manchester, of Ryde, Isle of Wight.
LAWSON, Donald, B.Sc. Glasgow, Bearsden, By Glasgow, Scotland.
LORD, Percival George, of Beckenham, Kent.
MCKELVEY, William Leslie, of Belfast, N. Ireland.
MARTIN, Geoffrey Phillip, B.Sc.(Civil) Birmingham, A.M.I.C.E., Brighton, Sussex.
MITCHELL, John Phillip, M.Sc. Leeds, B.Sc.(Hons.), of Liverpool.
MUKHERJEE, Kunjalal, B.E.(Civil) Calcutta, of Calcutta, India.
NIGHTINGALE, Alan Francis, of Lower Hutt, New Zealand.
PARTON, Girvan Meir, B.Sc.(Tech.) Manchester, A.M.I.C.E., of Newcastle, Staffs.
REVESZ, Stephen, M.Sc.(Eng.) London, D.I.C., A.M.I.C.E., of London.
ROBSON, John Rowland, of Bramhall, Cheshire.
SARKAR, Suniti, of Fort, Bombay.
SHUNN, William Gordon, B.Sc.(Eng.) Rand, of Johannesburg, South Africa.
SHUTTLEWORTH, Bernard Thomas, of Bolton, Lancs.
SYMES, Robert Francis, of London.
THOMASON, Dennis William, of Johannesburg, South Africa.
THOMPSON, Donald, of Urmston, Nr. Manchester.
THOMSON, Andrew Gordon, of London.
TONGE, Brian Yardley, of Farnworth, Lancs.
TONNER, Eric William, of Auckland, New Zealand.
TURNER, Colin Francis, of Cuddington, Nr. Northwich, Cheshire.
VIJIARATNAM, Arumugam, of Singapore.
WATSON, John Rex, of Wraxhall Somerset.
WEHRLE, John Emile Charles, of Stanmore, Middlesex.
WHITE, Allan Robert, of New Barnet, Herts.
WILKINS, John Richard, of London.
WILKINSON, Herbert, of Grappenhall, Nr. Warrington, Lancs.

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, December 15th, 1955

Ordinary General Meeting for the election of members at 5.55 p.m., followed by a Joint Meeting with the British Section of the Societe des Ingenieurs Civils de France, when M. R. Giguet will give a paper on " The Tignes Hydro-Electric Scheme."

Thursday, January 12th, 1956

Ordinary Meeting at 6 p.m., when Dr. R. K. Livesley, M.A., Ph.D., will give a paper on " The Application of an Electronic Digital Computer to Some Problems of Structural Analysis."

Thursday, January 26th, 1956

Ordinary General Meeting for the election of members at 5.55 p.m. followed by an Ordinary Meeting at 6 p.m., when Mr. F. M. Bowen, M.I.C.E., Assoc.I.Mech.E. (Member of Council), and Mr. B. E. S. Ranger, A.M.I.C.E. (Associate Member), will give a paper on " Steelwork in Southampton Docks Transit Shed 102."

Thursday, February 9th, 1956

Ordinary Meeting at 6 p.m., when Mr. T. Whitley Moran, B.A., B.A.I., M.I.C.E. (Member), will give a paper on " The Use of Gunite as a Structural Material."

Thursday, February 23rd, 1956

Ordinary General Meeting for the election of members at 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when three short papers on " Structural Aspects of Road Design " will be given as follows : Mr. P. L. Capper, T.D., M.Sc.(Eng.), A.M.I.C.E. (Member) on " Soil Mechanics " ; Dr. A. R. Collins, M.B.E., D.Sc., A.M.I.C.E. (Member of Council) on " Concrete Roads " ; Mr. A. W. Jarman, B.Sc., on " Flexible Road Construction."

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS, JANUARY 1956

The Examinations of the Institution will be held in all centres in the United Kingdom and Overseas on January 10th and 11th, 1956 (Graduateship) and January 12th and 13th (Associate Member).

EXAMINATIONS, JULY 1955—PRIZE AWARDS

In connection with the examinations held in July, 1955, the Council have made the following awards :

ANDREWS PRIZE (For the candidate who obtains the highest aggregate of marks in the Associate-Membership Examination, passing in all subjects)

John Leonard DARLISON, of Leyton.

HUSBAND PRIZE (For the candidate who takes the whole of the Associate-Membership Examination, passed in all subjects and obtains the highest marks in the paper " Structural Engineering Design and Drawing ")

John Dennis PINKNEY, of Scunthorpe.

WALLACE PREMIUM (SENIOR) (For the candidate who takes the whole of the Associate-Membership Examination, passes in all subjects, and obtains the highest marks in the paper " Theory of Structures (Advanced))"

John Leonard DARLISON, of Leyton.

WALLACE PREMIUM (Junior) (For the most successful candidate in the Graduateship Examination, passing in all subjects)

Frank Birkinshaw FAWCETT, of Ealand, Nr. Scunthorpe.

MACLACHLAN LECTURE COMPETITION, 1956

The closing date for the receipt of entries for the next Maclachlan Lecture Competition is Friday, March 30th, 1956. The general conditions of the competition are as follows :

1. The Institution of Structural Engineers shall institute a written lecture to be known as the Maclachlan Lecture and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Engineering as long as in every second year the subject shall be confined to steel structures.

3. Entrance into the competition for the Lecture shall be confined to Graduates and Associate Members of the Institution who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council

of the Institution, and all such papers (including the prize-winning Lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award, or for any other reason, the Institution shall transfer the above sums to the Research Fund of the Institution.

PARTICULARS OF THE COMPETITION FOR 1956

1. The Maclachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1956.

2. The subject of the Lecture shall be on any aspect of structural engineering.

3. The work should be submitted as the script of a lecture which the author, if successful in the competition, will deliver before an audience in the course of about one hour. The development of mathematical formulæ and detailed calculations should be avoided as far as possible in the text; if they are essential they should be embodied in appendices. Photographs, drawings, graphs, etc., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.

4. Six copies of each Lecture should be submitted and should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the Institution is Friday, March 30th, 1956.

LONDON GRADUATES' AND STUDENTS' SECTION

The next meeting of the Section will be held at 11, Upper Belgrave Street, London, S.W.1, on Monday, December 19th, 1955, when Mr. G. A. Wilson, M.Eng., M.I.C.E., M.I.Mech.E., will talk on "Engineering Problems in the Docks."

A meeting will be held on Wednesday, January 4th, 1956, when two speakers will address the Section on "Engineering in Europe and America."

Hon. Secretary: D. E. Capelin, 40, Thornton Crescent, Old Coulsdon, Surrey.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged:

Thursday, December 8th, 1955

At the College of Technology, Manchester. Mr. F. M. Little, B.Sc.(Tech.), A.M.I.C.E., A.M.I.Struct.E., on "The Influence of the Motor Vehicle on Urban Buildings."

Tuesday, January 10th, 1956

Joint Meeting with the Liverpool Branch of the Institute of Welding, at the University of Liverpool. Professor W. J. Kearton, D.Eng., M.I.N.A., M.I.Mech.E., on "Photoelasticity."

Thursday, January 12th, 1956

At the College of Technology, Manchester. Mr. Felix J. Samuely, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E. on "Space Frames and Skin Structures."

Wednesday, January 18th, 1956

At the Liverpool Engineering Society, Dale Street, Liverpool. Mr. A. Bolton, M.Sc.(Tech.) (Graduate) on "The Calculations of Critical Loads for Rigid Frames."

Friday, February 3rd, 1956

Annual Dinner and Dance at the Midland Hotel, Manchester.

Monday, February 13th, 1956

At the College of Technology, Manchester. Mr. T. Whitley Moran, B.A., B.A.I., M.I.C.E., M.I.Struct.E. on "The Use of Gunite as a Structural Material."

Monday, February 20th, 1956

The above meeting will be repeated at the Liverpool Engineering Society, Liverpool.

Meetings will commence at 6.30 p.m., preceded by tea at 5.45 p.m.

Joint Hon. Secretaries: J. L. Robinson, A.M.I.Struct.E., 314, Northenden Road, Sale, Manchester; M. D. Woods, 58, Spring Gardens, Salford, 6.

MIDLAND COUNTIES BRANCH

It has been necessary to make a number of changes in the arrangements for meetings as announced in the Sessional Programme. The following are the revised arrangements:

Wednesday, December 7th, 1955

A meeting will be held in the Lecture Room at the Wolverhampton and Staffordshire Technical College, Wulfruma Street, Wolverhampton, at 7 p.m., when Mr. G. McK. Thomson, A.M.I.C.E., A.M.I.Struct.E. will give a paper on "Hams Hall 'C' Generating Station."

Friday, January 27th, 1956

At the James Watt Memorial Institute, Birmingham. Mr. P. M. Worthington, B.Sc., on "The Design of Crane Gantries and the Action of Cranes on the Tracks."

Friday, February 24th, 1956

At the James Watt Memorial Institute, Birmingham. Mr. R. Howl, A.M.I.Struct.E., on "Welded Steel Road Bridge of Rigid Frame Construction at Walsall."

Friday, March 23rd, 1956

At the James Watt Memorial Institute, Birmingham. Mr. T. M. Charlton, M.A.(Cantab.), B.Sc.(Eng.) London on "Some Aspects of the Analysis and Design of Structures with particular reference to the Plastic Theory."

Hon. Secretary: L. A. Firminger, A.M.I.Struct.E., 190, Green Lanes, Sutton Coldfield, Nr. Birmingham, Warwicks.

MIDLAND COUNTIES' GRADUATES' AND STUDENTS' SECTION

Hon. Secretary: J. E. Jeffries, 18, Radnor Road, Handsworth, Birmingham 20.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged:

Tuesday, December 6th, 1955

Meeting at Middlesbrough.

Wednesday, December 7th, 1955
Meeting at Newcastle.

Wednesday, January 4th, 1956
Joint Meeting with the Northern Architectural Association at Newcastle. Paper by Mr. D. W. Cooper, B.Sc., A.M.I.Struct.E.

Wednesday, January 11th, 1956
Joint Meeting with The Institution of Civil Engineers, at Middlesbrough. Mr. D. R. R. Dick, B.Sc., M.I.C.E., on "The Civil Engineer and Britain's Atomic Factories."

Thursday, January 31st, 1956
At Middlesbrough. Mr. F. W. Slatter, M.I.Struct.E. and Mr. A. Brown, A.M.I.Mech.E., on "Foundations, Underpinning and Structural Problems at the *Daily News* Building, London."

Friday, February 1st, 1956
The above meeting will be repeated at Newcastle.

Middlesbrough meetings will be held at the Cleveland Scientific and Technical Institution, and Newcastle Meetings at the Neville Hall, Westgate Road. Meetings will commence at 6.30 p.m. preceded by buffet tea at 6 p.m.

Hon. Secretary: O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged:

Tuesday, December 6th, 1955
Mr. C. Munro, A.R.I.B.A., on "The Architect and the Structural Engineer."

Tuesday, January 3rd, 1956
Lecture to be arranged.

Tuesday, February 7th, 1956
Mr. J. H. Simpson, B.Sc., A.M.I.C.E., A.M.I.W.E., on "A New Bridge at Enniskillen with Prestressed Concrete Deck Beams."

Meetings will be held at the College of Technology, Belfast, at 6.45 p.m., and will be preceded by tea at the Overseas League premises, Wellington Place, Belfast, at 6 p.m.

Hon. Secretary: A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., "Barbizon," 2, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged:

Tuesday, December 6th, 1955
Mr. Donovan H. Lee, B.Sc.(Eng.), M.I.C.E., M.I.Mech.E., M.I.Struct.E., M.Am.Soc.C.E., on "The Advantages and Disadvantages of Structural Steelwork, Reinforced Concrete and Prestressed Concrete."

Tuesday, January 17th, 1956
Mr. D. V. Pike, M.I.Struct.E., A.M.I.C.E., on "Aluminium Alloy Structures."

Friday, February 10th, 1956
Joint Meeting with the Glasgow and West of Scotland Association of The Institution of Civil Engineers. Mr. D. A. Stewart, M.B.E., A.M.I.C.E., A.M.I.E.E., on "Survey of Modern Concrete Technique, Quality and Control."

Meetings will be held at the Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, at 7 p.m.

Hon. Secretary: W. Basil Scott, M.I.Struct.E., 19, Waterloo Street, Glasgow, C.2.

SOUTH WESTERN COUNTIES BRANCH

The following meetings have been arranged:

Friday, December 16, 1955
Meeting at Exeter, when a paper on Timber Construction will be given.

Friday, January 20th, 1956
At the Duke of Cornwall Hotel, Plymouth, 7 p.m. A paper on "The Development of Plastic Design," by Professor J. F. Baker, O.B.E., M.A., D.Sc., M.I.C.E., M.I.Struct.E. will be read by Mr. M. R. Horne, M.A., Ph.D., A.M.I.C.E. on behalf of the author.

Friday, February 17th, 1956
At the Duke of Cornwall Hotel, Plymouth, 7 p.m. Mr. L. E. Kent, B.Sc.(Eng.), M.I.C.E. (Vice-President), will give a paper on a subject to be announced later.

Joint Hon. Secretaries: E. W. Howells, M.I.Struct.E., 10/12, Market Street, Torquay, Devon; C. J. Woodrow, J.P., "Elstow," Hartley Park Villas, Tavistock Road, Plymouth.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged:

Thursday, December 15th, 1955
Joint Meeting with the Institution of Civil Engineers, at the South Wales Institute of Engineers, Cardiff, 6.30 p.m. Mr. F. Walby, M.I.C.E., and Mr. H. C. Adams, M.I.C.E. on "Prestressed Concrete as Applied to Building Frames."

Wednesday, January 18th, 1956
At the Mackworth Hotel, Swansea, 6.30 p.m. Major A. F. Allen, M.I.C.E., on "Description of Neath By-Pass Constructional Work."

Tuesday, February 14th, 1956
Joint Meeting with the Institute of Welding, at the South Wales Institute of Engineers, Cardiff. Mr. J. M. Burke, B.Sc., on "The New South Stand at Cardiff Arms Park."

Hon. Secretary: K. J. Stewart, A.M.I.C.E., A.M.I.Struct.E., 15, Glanmor Road, Swansea, Glam.

WESTERN COUNTIES BRANCH

The following meetings have been arranged:

Friday, December 2nd, 1955
Papers by Branch members: Mr. J. A. Young, A.M.I.C.E., A.M.I.Struct.E. on "Joints in Water Retaining Structures with particular reference to Reservoirs"; and Mr. N. T. A. Beavan, A.M.I.Struct.E. on "Co-operation between the Consultant and the Steelwork Engineer."

Friday, December 9th, 1955
Combined Dance at the Royal Hotel, Bristol.

Friday, January 6th, 1956
Mr. H. W. Moore, M.I.Struct.E. will give a paper on a subject to be announced later.

Friday, February 3rd, 1956
Mr. Hubert A. Mackrill, M.I.C.E., M.I.W.E., F.R.S.A., will read a paper in connection with the reinforced concrete construction on three local con-

tracts, namely, the new Engineering School, University of Bristol, the Water Tower, Durdham Downs and Messrs. Pilkington's Factory, Bristol.

Wednesday, February 15th, 1956
Annual Dinner, Royal Hotel, Bristol.

Unless otherwise stated, meetings will be held in the Small Lecture Theatre in the new Engineering School, University of Bristol, at 6 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary: E. Hughes, M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The following meetings have been arranged :

Wednesday, December 21st, 1955
Mr. J. Robertshaw, B.Sc., A.M.I.C.E., on " Geophysical Methods of Exploration and their application to Structural Engineering Problems."

Wednesday, January 11th, 1956
Joint Meeting with the Yorkshire Association of the Institution of Civil Engineers at the Blue Bell Hotel, Scunthorpe, at 6.30 p.m. Mr. G. M. Boyd, A.M.I.Struct.E. on " Brittle Fracture Problems in Steel Construction."

Wednesday, January 18th, 1956
Details to be arranged.

Wednesday, February, 1956
Details to be arranged.

Hon. Secretary: E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary: A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box No. 3306, Johannesburg, South Africa.

During weekdays Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. Phone 34-1111, Ext. 257.

Natal Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co. Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary: R. Stubbs, M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

PAPERS FOR PUBLICATION

The Literature Committee would be glad to consider offers of papers for presentation at the Institution or for publication in the Journal.

The following is a summary of the Committee's requirements relating to articles and papers : a copy of the full conditions may be obtained from the Secretary.

(1) Articles must be of an appropriate character, having a bearing upon structural engineering or upon some kindred scientific or constructional subject, and must be approved by the Literature Committee. A short title is an advantage.

(2) Contributions must be original either in subject-matter or in presentation. Articles which have already been published or have been read to other organised bodies, or are carelessly prepared, will not be accepted for publication.

(3) The style of writing will necessarily vary with the individual, but authors are requested to write as plainly and simply as their subject will allow. Papers should be written in the third person.

(4) Where the subject allows, a brief introduction or synopsis should state clearly the purpose and scope of the paper or article, and the author's conclusions or recommendations should be summarised at the end of the paper.

In order to facilitate the indexing of articles for reference, the author will be required in addition to prepare a short precis not exceeding 25 words for inclusion under the title of the paper on the contents page of the Journal.

(5) Illustrations are desirable where they assist in explaining the context or are fundamental to the subject. They should not be used if unnecessary for these purposes. Illustrations may be either line drawings or photographs.

(6) Line drawings must be specially prepared for reproduction on smooth white paper, or clear tracing paper, with heavy main lines and large clear lettering drawn in Indian ink with a mapping pen. Alternatively, the author may submit drawings on one sheet of paper with the relevant lettering on a cover sheet of tracing paper.

The printed page of THE STRUCTURAL ENGINEER is 7 in. wide by 10 in. deep. The drawings, where practicable, should be prepared not larger than twice this size with a view to half-scale reproduction. Unavoidably large drawings which require reduction to one-third size or less, must be specially heavy and with proportionately large lettering for clear reproduction. Ordinary working drawings are not satisfactory.

(7) Where photographs are submitted they should be printed *black on glossy paper*.

(8) MS. typewritten in double spacing should be submitted in duplicate.

Brevity is an advantage and papers should not normally exceed 7,500 words in length.

REPRINTS FROM " THE STRUCTURAL ENGINEER "

Authors who wish to obtain reprints of their papers published in " The Structural Engineer " should notify the Secretary of the Institution not later than the 15th of the month in which the paper is published.

EXAMINATIONS

Preparation for the Examinations of the Institution by Attendance at Technical Colleges

A candidate for Graduateship or Associate-Membership may be able to attend a technical college ; these notes are intended to guide him in choosing the most suitable instruction.

Preparation for the Graduateship Examination

Technical Colleges offer :

(a) Full-time courses for degrees or Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At technical colleges courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects

from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

Preparation for the Associate-Membership Examination

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering; these cover only part of the requirements for the Associate-Membership Examination.

Colleges in List "A" provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

List "A"

Bath Technical College.
Belfast College of Technology.
Birmingham College of Technology.
Bolton Municipal Technical College.
Bradford Technical College.
Bridgend Technical College.
Chesterfield College of Technology.
Derby Technical College.
Dudley and Staffordshire Technical College.
Glasgow Royal Technical College.
City of Liverpool College of Building.
L.C.C. Brixton School of Building, S.W.4.
L.C.C. Hammersmith School of Building and Arts and Crafts, W.12.
Manchester College of Technology.
Middlesbrough, Constantine Technical College.
Nottingham and District Technical College.
Salford, Royal Technical College.
South-East London Technical College, Lewisham Way, S.E.4.
South-West Essex Technical College, Walthamstow, E.17.
Stafford, County Technical College.
Stockport College for Further Education.
Twickenham Technical College.
Willesden Technical College, N.W.10.

Colleges in List " B " provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete.

List " B "

Brighton Technical College.
Cardiff Technical College.
Edinburgh, Heriot-Watt College.
Huddersfield Technical College.
Leeds College of Technology.
London, Battersea Polytechnic, S.W.11.
London, Northampton Polytechnic, E.C.1.
L.C.C. Westminster Technical College, S.W.1.
Newcastle upon Tyne, Rutherford College of Technology.
Plymouth and Devonport Technical College.
Preston, Harris Institute.
Rotherham College of Technology.
Wigan Mining and Technical College.
Woolwich Polytechnic, S.E.18.
West Ham College of Technology.

Students are advised to take the organised courses in Structural Engineering where these are available.

Book Reviews

Analysis of Symmetric Cylindrical Shells. Its Application to Civil Engineering Design, by John McNamee. (London : H.M.S.O., 1955.) 9½ in. × 7½ in. 84 pp. plus Appendix. Price 12s. 6d.

This new Building Research report, based on lectures which were originally given at Liverpool University, gives an account of the theory of thin cylindrical shells for those who are approaching the subject for the first time. The Author has developed the theory of shells by using (and generalizing) concepts of beam and slab analysis with which engineers may be presumed to be familiar, and the application of the theory to the stress-analysis of reinforced concrete structures is illustrated by detailed numerical examples.

The report provides a comprehensive account of the elements of the theory, and the analysis is carried up to the point where it can be used to determine numerical values for the stresses in shell roofs.

Forages et Sondages, by H. Cambefort. (Paris : Eyrolles, 1955.) 6 in. × 10 in., 400 pages, 370 figures and diagrams. Price Frs. 3,200.

All Engineers and Contractors interested in the sinking of borings, prospecting, and geotechnical processes should find this treatise of considerable value to them.

The sinking of borings is of paramount interest to people concerned with obtaining oil and other treasures hidden beneath the ground, often at very great depths, below strata of hard rock. The book presents the reader with many problems of this nature and describes various methods of solving the difficulties.

The subject matter is well presented and the book contains a large number of excellent illustrations. A special feature is the extensive description of the plant used for sinking borings and clear illustrations of some of the special devices employed for boring through hard rock.

The author describes the processes of chemical consolidation of ground, electro-osmosis and other means of improving the stability of ground so that it forms a satisfactory bearing stratum. Interesting illustrations of works on which these processes were employed are included in the text and are of great value to all Engineers who are concerned with the construction of foundations on difficult sites where existing works have to be maintained.

The specialist who is concerned with research on these matters will also find interesting data obtained in the laboratory as well as on the site.

P.J.G.